STINSON

Investigation of a Highway Bridge

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INVESTIGATION OF A HIGHWAY BRIDGE

BY

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THIS IS TO CERTIFY THAT THE THESIS PREPARED UNDER MY SUPERVISION BY

SPENCER A STINSON

ENTITLED INVESTIGATION OF A HIGHWAY BRIDGE

IS APPROVED BY ME AS FULFILLING THIS PART OF THE REQUIREMENTS FOR THE

DEGREE OF Bachelor of Science in Civil Engineering

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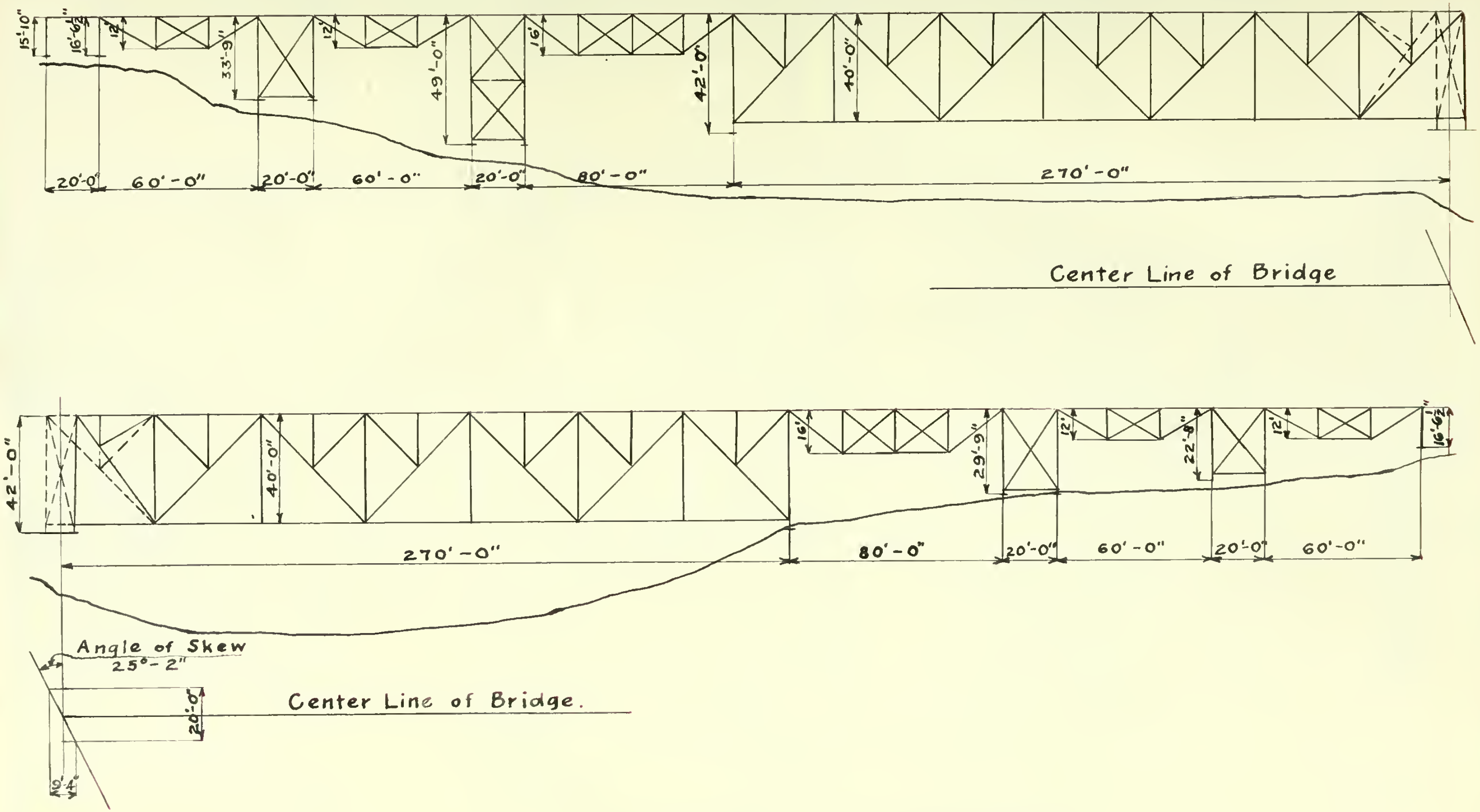


GENERAL DIMENSIONS

INVESTIGATION OF A HIGHWAY BRIDGE.

PART I.

Article 1-Introduction.

The bridge which is the subject of investigation in the following thesis is located in the city of Danville, Vermillion County, Illinois. It forms the Gilbert Street crossing of the Vermillion river, and is about one half mile west of the center of the business section.

The bridge was erected in 1897 by the Chicago Bridge and Iron Works. Through the courtesy of this company detail drawings were secured for use in computing the weights of the different units the summary of which will be given in the tables of Article 4. These tables will be used for determining the dead loads. The live load for which the bridge was designed will be determined in Article 5. In computing the weights ordinary methods of bridge analysis will be used. The positive sign will be used to denote tension and the negative sign to denote compression.

Article 2-Object of Thesis.

The object of this thesis will be to examine the design and determine the efficiency of the parts of the structure according to Cooper's Specifications for Steel Highway Bridges, edition of 1901.

Article 3-Description of Structure.

The structure under investigation is a deck bridge consisting of two 270 foot spans, two 80 foot spans, four 60 foot spans, four towers and three bents. The general elevation of the structure is

given in Plate I. The trusses are spaced 20 feet center to center, the roadway is 20 feet in the clear and there are two overhanging footwalks each one being 6 feet wide in the clear.

The trusses of the 270 foot spans are of the subdivided Warren type. The channel pier which supports one end of each 270 foot span is on a skew, the angle of skew being 25 degrees and 2 minutes. This makes the two trusses of each long span unequal in length, the difference being 9 feet and 4 inches. This difference is accomodated by shortening the last panel of one truss. The main parts of some of the members of the shorter truss are built of somewhat lighter material than the corresponding members of the longer truss but the detailing and connections are the same. These trusses are 40 feet deep center to center of chords.

The trusses of the 80 and 60 foot spans are of the Pratt type, the depths being 16 feet and 12 feet center to center of chords respectively.

The four towers form the intermediate supports for the 80 and 60 foot spans. Each tower is 20 feet long. Plates will be used to give the elevations and plans of the different trusses, etc., together with the stresses and the composition of the members.

Article 4-Summery of Weights.

Table I-270 Foot Span.

Steel

Ref. No.	Name of Member	Total Weight: Main Members	Total Weight: Details	Details as percent of Main Member
	Long Truss			
1	End Posts	6,066	6,366	105.0
2	Top Chord	20,406	6,675	32.7
3	Lower Chord	15,233	1,194	7.8
4	Inclined Struts	15,280	7,118	46.6
5	Intermediate Posts	3,416	3,326	97.4
6	Sub Verticals	2,660	1,660	62.3
7	Main Ties	8,817		
8	Sub Ties	1,309		
9	Suspenders	336		
	Total	73,523	26,339	35.8
	Short Truss			
1	End Posts	6,066	6,366	105.0
2	Top Chord	18,583	6,551	35.3
3	Lower Chord	14,171	975	6.9
4	Inclined Struts	14,850	7,012	47.2
5	Intermediate Posts	3,416	3,326	97.4
6	Sub Verticals	2,532	1,627	64.3
7	Main Ties	6.022		
8	Sub Ties	1,282		
9	Suspenders	336		
	Total	67,258	25,857	38.5

Table I-Continued.

Ref. No.	Name of Member	Total Weight		Details as percent of Main Member
		Main Members	Details	
10	Floor Beams	22,913		
11	Hand Rail		8,299	
12	Vibration Struts	6,550	4,822	73.6
13	Top Lateral Rods	3,458		
14	Bottom " "	3,669	153	4.2
15	Wind Bracing	6,200	286	4.6
16	Wing Plates		1,211	
17	Chord Pins & Nuts		4,113	
18	Pedestals		5,490	
19	Bolts & Washers		495	
20	Spikes		520	
	Total	42,790	25,389	59.3

Weight of Long Truss = 99,862 pounds

" " Short " = 93,115 "

" " Laterals etc.= 68,179 "

Total Weight of Metal = 261,156 pounds.

Lumber

Ref. No.	Name of Member	Weight
1	Flooring	97,050
2	Joists	90,500
3	Felloe Guards	9,800
	Total	197,350

Table II-80 Foot Span.

Steel

Ref. No.	Name of Member	Total Weight		Details as percent of Main Member
		Main Members	Details	
1	Top Chord	4,936	2,525	51.2
2	Lower Chord	1,520		
3	Intermediate Posts	2,154	1,770	82.2
4	Main Ties	3,840		
5	Counters	309	18	5.8
6	Floor Beams	8,120		
7	Hand Rail		2,503	
8	Top Lateral Rods	805		
9	Bottom " Struts	1,138	343	30.2
10	Wind Bracing	539	39	7.2
11	Wing Plates		197	
12	Chord Pins & Nuts		600	
13	Rollers		139	
14	Bolts & Washers		165	
15	Spikes		170	
	Total	23,361	8,469	36.3

Grand Total Weight of Metal = 31,830 pounds.

Lumber

Ref. No.	Name of Member	Weight
1	Flooring	28,850
2	Joists	25,860
3	Felloe Guards	2,880
	Total	57,590

Table III-60 Foot Span.

Steel

Ref. No.	Name of Member	Total Weight		Details as percent of Main Member
		Main Members	Details	
1	Top Chord	2,570	1,610	62.6
2	Lower Chord	700		
3	Intermediate Posts	816	868	106.0
4	Main Ties	1,740		
5	Counters	630	36	5.7
6	Floor Beams	6,500		
7	Hand Rail	385	13	3.4
8	Top Lateral Rods	501		
9	Bottom " Struts	756	179	23.7
10	Wind Bracing	430	32	
11	Wing Plates		175	7.4
12	Chord Pins & Nuts		428	
13	Bolts & Washers		186	
14	Spikes		130	
	Total	15,028	3,657	24.3

Grand Total Weight of Metal = 18,685 pounds.

Lumber

Ref. No.	Name of Member	Weight
1	Flooring	20,844
2	Joists	19,400
3	Felloe Guards	2,310
	Total	42,554

Table IV-Tower No. 1.

Steel

Ref. No.	Name of Member	Total Weight		Details as percent of Main Member
		Main Members	Details	
1	Legs	4,650	5,509	118.4
2	Struts	3,431	1,224	35.7
3	Bracing	1,675	68	4.1
4	Pins	------	80	
	Total	9,756	6,881	70.5

Grand Total Weight of Metal = 16,637 pounds.

Lumber

Ref. No.	Name of Member	Weight
1	Flooring	6,660
2	Joists	6,470
3	Felloe Guards	864
	Total	13,994

Table V-Tower N. 2.

Steel

Ref. No.	Name of Member	Total Weight: Main Member	Total Weight: Details	Details as percent of Main Member
1	Legs	7,540	7,513	99.6
2	Struts	4,653	2,073	44.5
3	Bracing	2,744	136	5.0
4	Pins	______	160	
	Total	14,937	9,882	66.1

Grand Total Weight of Metal = 24,819 pounds.

Lumber		
Ref. No.	Name of Member	Weight
1	Flooring	6,660
2	Joists	6,470
3	Felloe Guards	864
	Total	13,994

Table VI-Tower No. 3.

Steel

Ref. No.	Name of Member	Total Weight		Details as percent of Main Member
		Main Member	Details	
1	Legs	4,570	4,867	106.6
2	Struts	2,490	1,071	43.0
3	Bracing	1,760	76	4.3
4	Pins		80	
	Total	8,820	6,094	69.1

Grand Total Weight of Metal = 14,914 pounds.

Lumber

Ref. No.	Name of Member	Weight
1	Flooring	6,660
2	Joists	6,470
3	Felloe Guards	864
	Total	13,994

Table VII-Tower No. 4.

Steel

Ref. No.	Name of Member	Total Weight		Details as percent of Main Member
		Main Member	Details	
1	Legs	2,540	4,670	183.6
2	Struts	3,398	1,140	33.5
3	Bracing	1,335	68	5.1
4	Pins		80	
	Total	7,273	5,958	81.9

Grand Total Weight of Metal = 13,231 pounds.

Lumber

Ref. No.	Name of Member	Weight
1	Flooring	6,640
2	Joists	6,470
3	Felloe Guards	864
	Total	13,994

Table VIII-Bent No. 1.

Steel

Ref. No.	Name of Member	Total Weight		Details as percent of Main Member
		Main Member	Details	
1	Legs	627	665	106.0
2	Struts	635	150	23.6
3	Floor Beam	1,625		
4	Bracing	241	16	6.6
5	Pins		35	
	Total	3,128	866	27.7

Grand Total Weight of Metal = 3,994 pounds.

Lumber

Ref. No.	Name of Member	Weight
1	Flooring	3,330
2	Joists	3,235
3	Felloe Guards	432
	Total	6,997

Table IX-Bents No. 2 & 3.

Steel

Ref. No.	Name of Member	Total Weight		Details as percent of Main Member
		Main Member	Details	
1	Legs	807	1,712	212.2
2	Struts	623	228	36.6
3	Bracing	248	16	6.4
4	Pins	———	20	
	Total	1,678	1,976	117.8

Grand Total Weight of Metal = 3,654 pounds.

Lumber

Ref. No.	Name of Member	Weight
1	Flooring	3,330
2	Joists	3,235
3	Felloe Guards	432
	Total	6,997

PART II.

EFFICIENCY OF THE MEMBERS.

Article 5-Determination of Classof Bridge.

It will be necessary to determine the live load for which the bridge was designed before computing the stresses in the members. To find this live load we will find the area of the cross-section in the bottom chord member L_0L_1 of the long truss of the 270 foot span in excess of the amount needed for the dead load stress, then compute the amount of live load stress this will carry, and finally compute the live load necessary to give this stress.

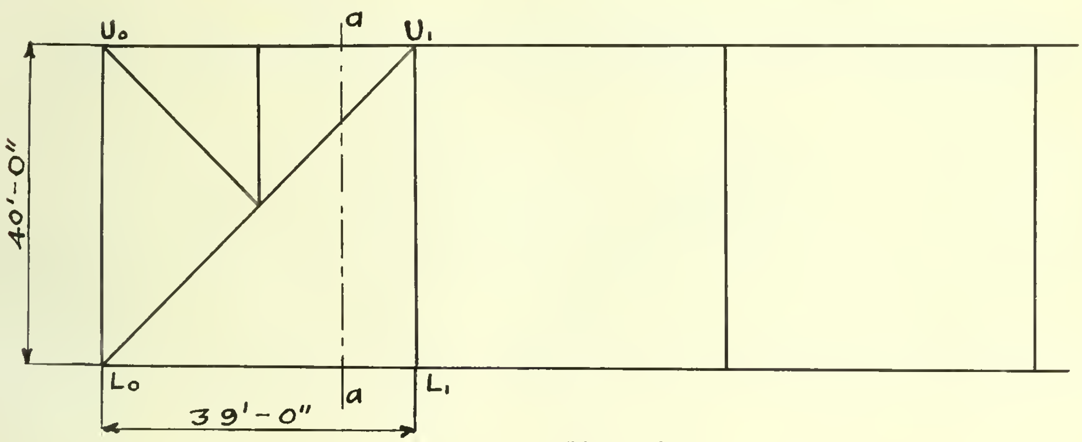


Fig. 1.

By refering to Table I it is seen that the weight of the long truss is 99,862 pounds, of the lateral system is 68,179 pounds, and of the lumber is 197,350 pounds. The total dead load supported by the long truss is therefore

$$99,862 + \frac{68,179}{2} + \frac{197,350}{2} = 232,627 \text{ pounds.}$$

As there are fourteen equal panels in the truss the load on each panel is 232,627 ÷ 14 = 16,620 pounds, and the dead load reaction is 232,627 ÷ 2 = 116,314 pounds.

Pass a section a-a as shown in Fig. 1 and take U_1 as a center of moments. Then the dead load moment in the section is

$$M = 116,314 \times 39.0 - 16,620 \times 19.5 - \frac{16,620}{2} \times 39.0 - L_0L_1 \times 40 = 0,$$

whence $L_0L_1 = + 97,400$ pounds.

As 25,000 pounds per square inch is the allowable unit stress for dead load, 97,400 ÷ 25,000 = 3.90 square inches is the required area for this dead load stress.

The member L_0L_1 is composed of two bars 5 x $1\frac{1}{4}$ inches in section, hence the actual area of the member is 2 x 6.25 = 12.5 square inches. The difference, 12.5 - 3.9 = 8.6 square inches, is the available area for live load. As the allowable live load stress per square inch is 12,500 pounds, the total live load stress allowable in the member is 8.6 x 12,500 = 107,500 pounds.

If R equals the live load reaction, and P equals the live panel load then R = 7P, and

$$M = 7P \times 39.0 - P \times 19.5 - \frac{P}{2} \times 39.0 - 107,500 \times 40 = 0,$$

whence P = 18,400 pounds. As the area of the floor contributing to the panel load is 19.5 x 16 square feet, this panel load would necessitate a load of 18,400 ÷ (19.5 x 16) = 59 pounds per square foot.

To determine the live load for which the floor was designed we will investigate the joists. The joists are 4 x 14 inch timbers spaced 2.46 feet center to center while the flooring is $2\frac{1}{2}$ inches thick. The section modulus of the joist is equal to $bd^2 \div 6$, where b is the breadth and d is the depth, then denoting the section modulus by c,

$$c = \frac{4 \times 14^2}{6} = 130,$$

as 1,200 pounds per square inch is the allowable stress in wood,

$M = 130 \times 1{,}200 = 156{,}000$ pound inches is the allowable moment in the beam.

Taking the weight of wood to be 4.5 pounds per board foot the moment in the beam due to dead load is,

$$M = \frac{1}{8}\left[(2.46 \times 2.5 \times 4.5) + (1.17 \times 4 \times 4.5)\right] \times 19.5^2 \times 12, \text{ or}$$

$M = 27{,}600$ pound inches, the panel length or span of the beam being 19.5 feet.

The moment available for live load is therefore equal to $156{,}000 - 27{,}600 = 128{,}400$ pound inches.

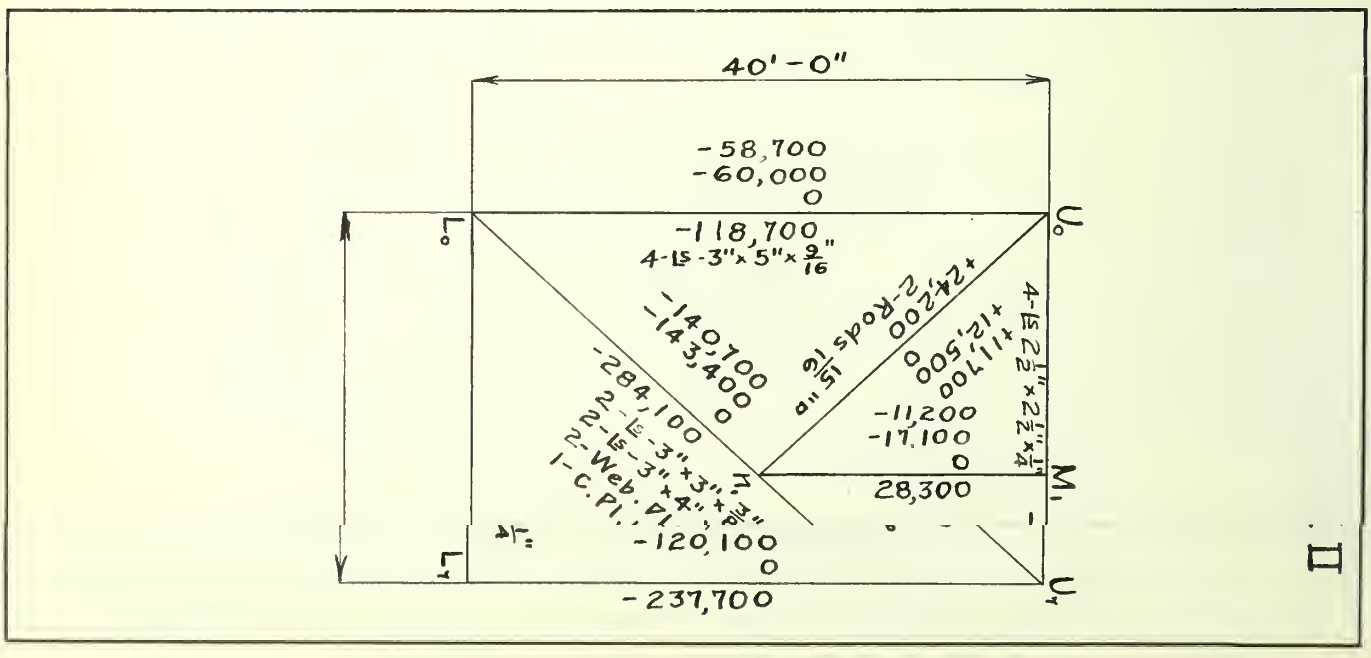
If we denote the live load per square foot by p we get

$$\frac{1}{8}(p \times 2.46) \times 19.5^2 \times 12 = 128{,}400$$

from which $p = 91$ pounds per square foot.

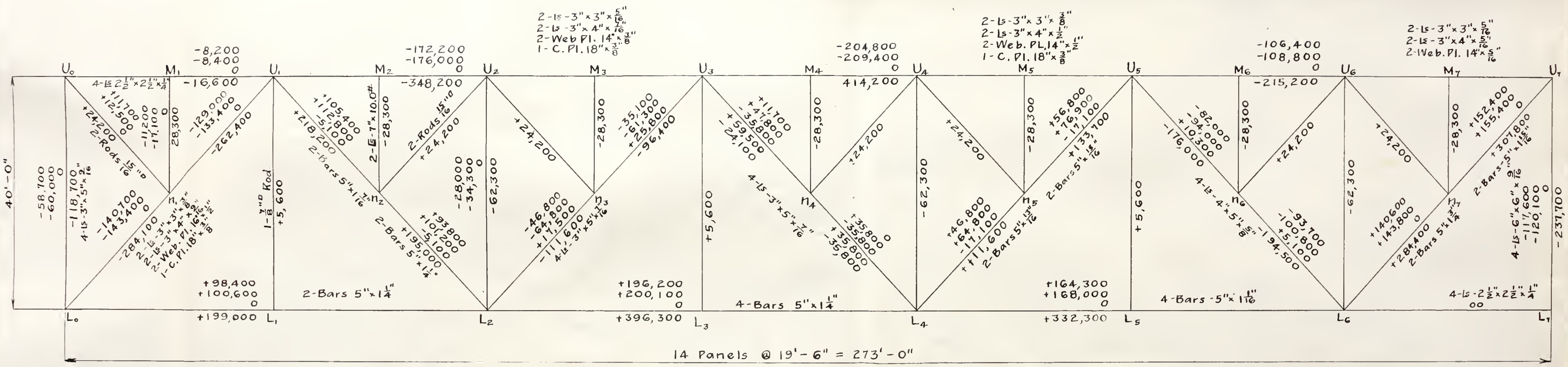
An examination of Table A in Cooper's specifications shows that these loads most nearly correspond to his class D for highway bridges. In class D the trusses for spans of 200 feet or over are designed for a live load of 55 pounds per square foot of floor surface and the floor and its supports are designed for a load of 80 pounds per square foot of floor surface, or 6 tons on two axles 10 feet centers.

These loads will be used for obtaining stresses throughout this investigation.

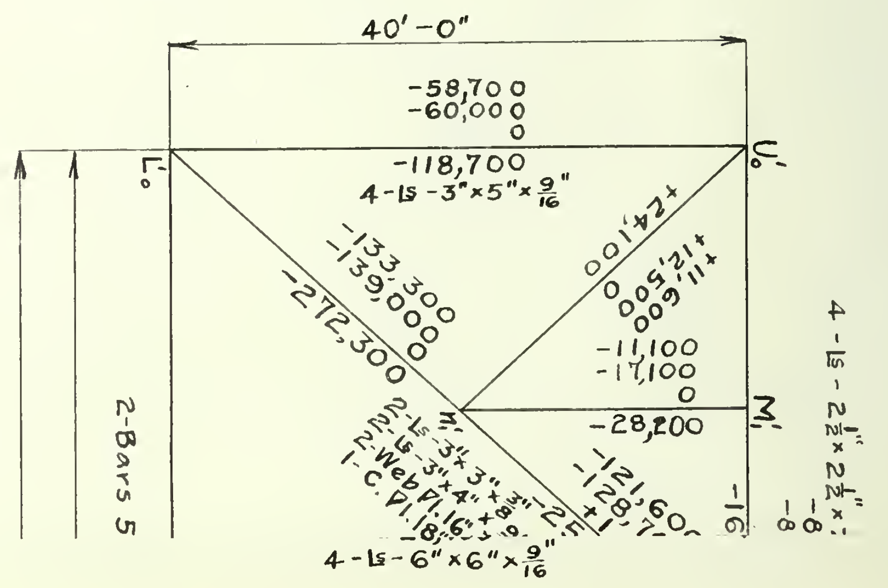


II
40'-0"
-58,700
-60,000
0
-118,700
4-Ls-3"x5"x 9/16"
L_0
U_0
M_1
U_1
L_1
+24,200
2-Rods 15/16"
+12,500
+11,700
0
4-Ls 2 1/2"x2 1/2"x 1/4"
-11,200
-17,100
0
28,300
-140,700
-143,400
0
-284,100
2-Ls-3"x3"x 3/8"
2-Ls-3"x4"x 3/8"
2-Web. Pl.
1-C.Pl.
-120,100
0
-237,700

PLATE II



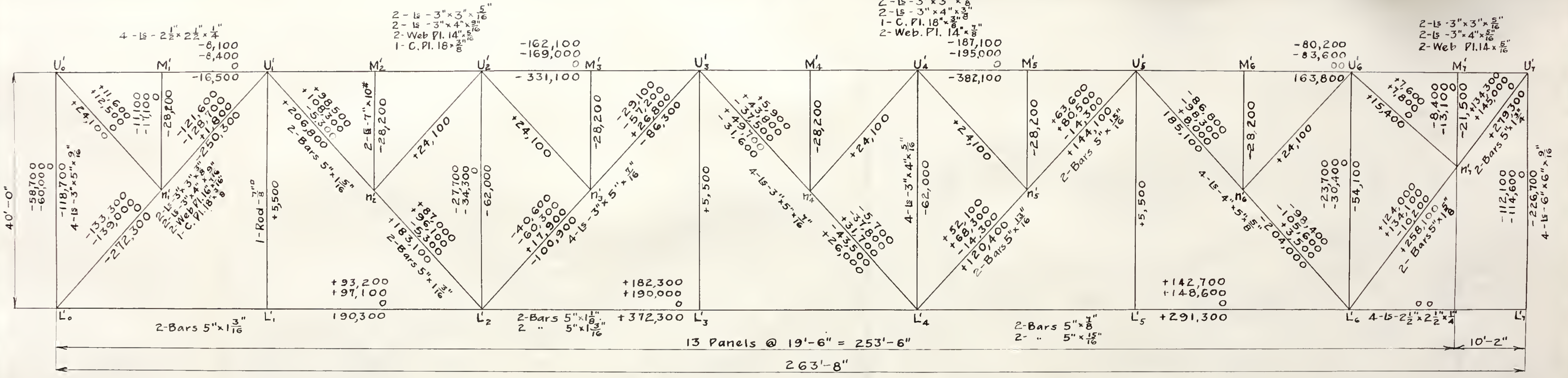
LONG TRUSS OF 270 FOOT SPAN



40′-0″
-58,700
-60,000
0
-118,700
4-Ls-3″×5″×9/16″
+24,100
+11,600
+12,500
0
-11,100
-17,100
0
-28,200
-133,300
-139,000
0
-272,300
-121,600
-16
4-Ls-2½″×2½″×
-8
-8
4-Ls-6″×6″×9/16″
2-Bars 5

II.

PLATE III.



SHORT TRUSS OF 270 FOOT SPAN

Article 6. 270 Foot Span.

Compression Members.

1. Intermediate Posts. All the intermediate posts are composed of four angles $3'' \times 4'' \times \frac{5''}{16}$ and the stresses are the same in all. The dead load stress is 28,000 pounds and the maximum live load stress is 34,300 pounds, both being compression. From the specifications we get the allowable dead load unit stress from the formula $P = 22,000 - 80\frac{l}{r}$ pounds per square inch, and the allowabla live load unit stress from $P = 11,000 - 40\frac{l}{r}$ pounds per square inch. The length l in this case is equal to $40 \times 12 = 480$ inches.

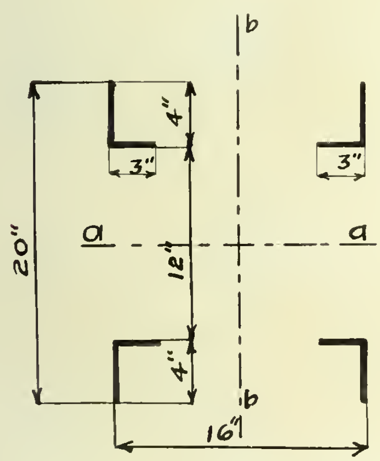


Fig. 2.

The total area of the section is $4 \times 2.09 = 8.36$ square inches. The moment of inertia of the section about the axis a-a is

$I = 4\left[3.38 + 2.09(6 + 1.26)^2\right]$ or

$I = 455$ inches^4. Then r about axis a-a equals $\sqrt{\frac{455}{8.36}} = 7.37$ inches.

The moment of inertia about the axis b-b is

$I = 4\left[3.38 + 2.09(8 - 0.76)^2\right] = 451$ inches^4 and $r = \sqrt{\frac{451}{8.36}} = 7.33$ inches.

Hence the allowable dead load unit stress is $22,000 - 80\frac{480}{7.33}$ $= 16,760$ pounds and the area required for dead load is

$$a_1 = \frac{28,000}{16,700} = 1.67 \text{ square inches.}$$

The allowable live load unit stress equal to $\frac{16,760}{2} = 8,380$ pounds and the area required for live load is

$a_2 = \frac{34,300}{8,380} = 4.09$ square inches. The total required

area is therefore 1.67 + 4.09 = 5.76 square inches and as the actual area is 8.36 square inches the efficiency of the member is

$$E = \frac{8.36}{5.76} = 1.45.$$

In the connections $\frac{3}{8}$-inch plates are used with $\frac{3}{4}$-inch rivets. The rivets are field riveted and in double shear. The allowable shearing stress is $\frac{2}{3}$ x 10,000 = 6,670 pounds per square inch, and the allowable bearing stress is $\frac{2}{3}$ x 18,000 = 12,000 pounds per square inch. From Carnegie's hand book we get the value of a $\frac{3}{4}$-inch rivet in single shear at 6,670 pounds per square inch to be $\frac{2}{3}$x4,420 = 2,950 pounds, in double shear it would be 2 x 2,950 = 5,900, while the bearing value of the same rivet in a $\frac{3}{8}$-inch plate at 12,000 pounds per square inch is 3,380 pounds. Hence the number of rivets needed inthe connection is $\frac{28,000 + 34,300}{3,380}$ = 18.4. There are actually however but 16 rivets and the efficiency is $\frac{16}{18.4}$ = 0.87.

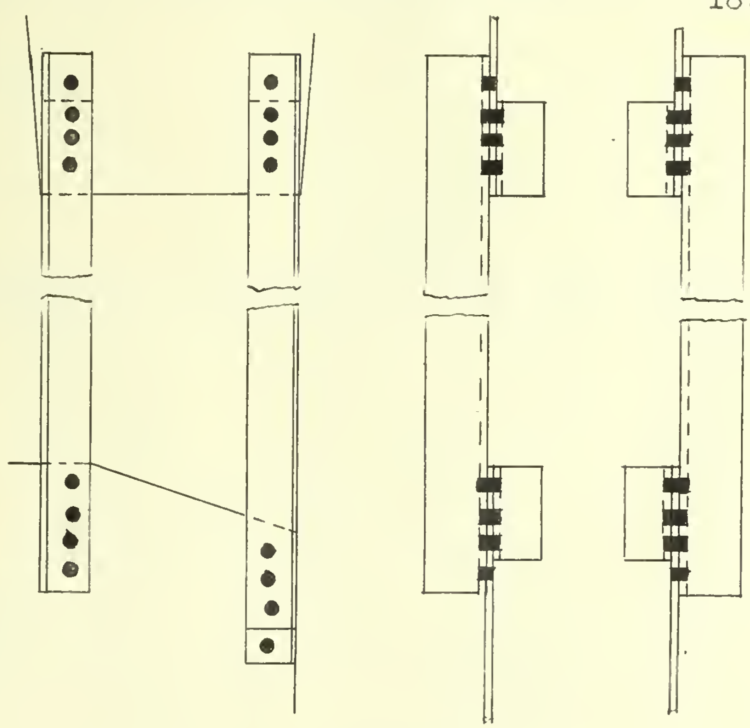


Fig. 3.

Intermediate Post Connections.

2. Sub-Verticals. Each sub-vertical is composed of two 7-inch, 100-pound channels and the stresses carried by them are 11,200 pounds due to dead load and 17,100 pounds due to live load. In the case of the alternative engine load the weight of the engine is but 12,000 pounds and since this is considerably less than the stress produced by the uniform live load it will not be considered in the investigation of the sub-verticals. As for the intermediate posts the allowable stresses are $P = 22,000 - 80\frac{l}{r}$ and $P = 11,000 - 40\frac{l}{r}$ pounds per square inch for dead and live load respectively.

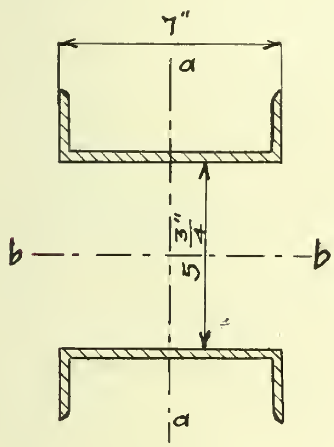


Fig. 4.

The channels of post $M_1 n_1$ are spaced 11 inches back to back. In all the other sub-verticals the spacing is $5\frac{3}{4}$ inches as shown in Fig. 4. About axis a-a the radius of gyration, from Carnegie's hand book, is 2.72 inches. About axis b-b the moment of inertia is

$$I = 2\left[0.98 + 2.85(2.875 + 0.546)^2\right] \text{ or}$$

$I = 68.8$ inches^4 and the radius of gyration is $r = \sqrt{\frac{68.8}{2 \times 2.85}} = 3.47$ inches.

As the length of the member is $19 \times 12 + 4\frac{5}{8} = 232.6$ inches, the allowable dead load stress is $P = 22,000 - 80 \times \frac{232.6}{2.72} = 15,150$ pounds per square inch, and the required dead load area is $\frac{11,200}{15,150} = 0.74$ square inches, while the allowable live load stress is $\frac{15,150}{2} = 7,575$ pounds per square inch and the required live load area is $\frac{17,100}{7,575} = 2.26$ square inches. The total required area is, therefore, $0.74 - 2.26 = 3.00$ square inches. The actual area is $2 \times 2.85 = 5.70$ square inches and therefore the efficiency is $\frac{5.70}{3.00} = 1.90$.

The upper end connections for all sub-verticals are the same

but at the lower end of those sub-verticals connecting to tension members, members composed of eye-bars, $\frac{5}{8}$-inch jaw plates are used, the pin being $5\frac{1}{8}$ inches in diameter; while at the lower end connections to compression, or built up members, the web of the channel is reinforced by one $\frac{1}{4}$-inch pin plate, the pin being only $2\frac{3}{8}$ inches in diameter. Fig. 5 illustrates the connections.

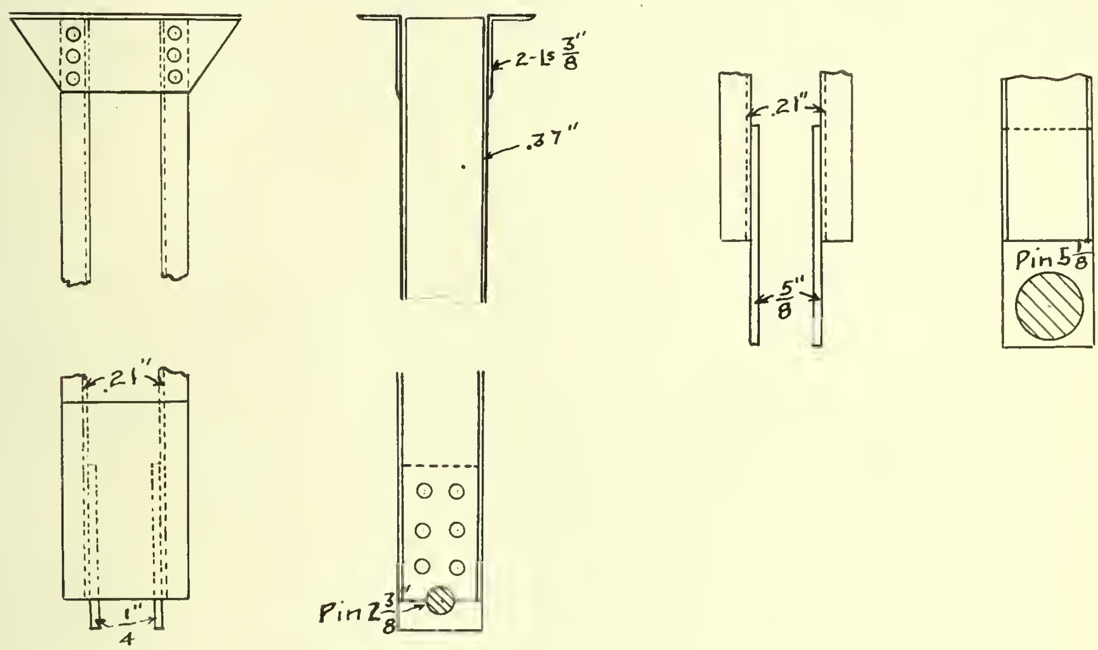


Fig. 5.

The rivets are $\frac{5}{8}$-inch shop and hence the allowable shear and bearing values are 10,000 and 18,000 pounds per square inch respectively. As the rivets are in single shear the shearing value is the limiting condition for the upper connection. The shearing value of a $\frac{5}{8}$-inch rivet at 10,000 pounds per square inch is 3,068 pounds and hence the required number of rivets is $\frac{28,300}{3,068} = 9.23$; as the actual number is 12 the efficiency is $\frac{12}{9.23} = 1.30$.

The web of the channel is but 0.21 inches in thickness and the bearing value of a $\frac{5}{8}$-inch rivet in the web is $\frac{5}{8} \times 0.21 \times 18,000$

= 2,360 pounds. Hence the required number of rivets for the connection to tension members shown by (b) of Fig. 5, is $\frac{28,300}{2,360} = 12.0$ as there are just 12 rivets in the connection the efficiency is 1.0.

The required thickness of pin plate for this connection is $\frac{28,300}{18,000 \times 5.01} = 0.313$ inches; as the actual thickness is $2 \times \frac{5}{8} = 1\frac{1}{4}$ inches the efficiency is $\frac{1.25}{0.313} = 3.98$.

In the connection shown by (a) of Fig. 5 the pin plates take $\frac{0.25}{0.25 - 0.21} \times 28,300 = 15,400$ pounds and this requires $\frac{15,400}{2,360} = 6.52$ rivets; as the actual number is 12 the efficiency is $\frac{12}{6.52} = 1.84$.

The required thickness of bearing plates is $\frac{28,300}{18,000 \times 2.38} = 0.662$ inches; the actual thickness is $2 \times (0.25 + 0.21) = 0.92$ inches and hence the efficiency is $\frac{0.92}{0.662} = 1.39$.

3(a). End Posts. Post L_0U_0. The end posts L_0U_0 serve as the supports for one end of the 80-foot truss and the greatest stresses in them are due principally to the reactions of this truss. The stresses in each post are - 58,700 pounds due to dead load, - 60,000 pounds due to live load, and - 35,700 pounds due to the overturning action of the wind.

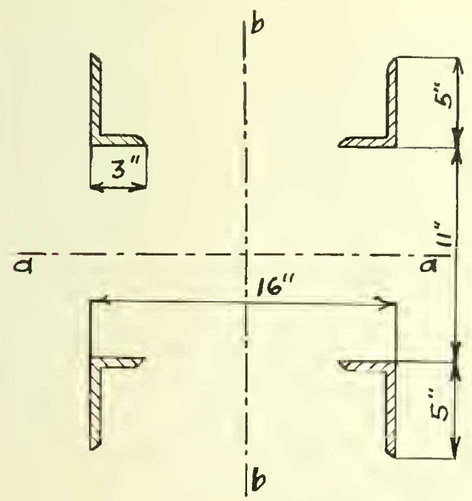


Fig. 6.

The wind stress is $\frac{35,700}{58,700 + 60,000} \times 100 = 30.1$ percent of the combined dead and live load stresses, and since this is greater than 25 percent, the wind load must be considered according to article 52 of the Specifications.

Each post L_0U_0 is composed of

four angles $5'' \times 3'' \times \frac{9''}{16}$, and the section is as shown in Fig. 6. The area of the section is $4 \times 4.19 = 16.76$ square inches. The moment of inertia about the axis a-a is, $I = 4\left[10.43 + 4.19(5.5 + 1.77)^2\right]$ $= 930$ inches^4. The radius of gyration is equal to $\sqrt{\frac{930}{16.76}} = 7.44$ inches.

The moment of inertia about the axis b-b is equal to $I = 4\left[2.83 + 4.19(8.0 - 0.77)^2\right] = 889$ inches^4 and the radius of gyration is equal to $\sqrt{\frac{889}{16.76}} = 7.28$ inches.

Therefore the allowable dead load stress is equal to $22{,}000 - 80 \times \frac{480}{7.28} = 16{,}700$ pounds per square inch and the allowable live load stress is equal to $\frac{16{,}700}{2} = 8{,}350$ pounds per square inch. The area required for the dead load stress is equal to $\frac{58{,}700}{16{,}700} = 3.51$ square inches and the area required for the live load stress is equal to $\frac{60{,}000}{8{,}350} = 7.12$ square inches.

The average allowable unit stress in the member due to dead, live, and wind loads is $1.25 \times \frac{58{,}700 + 60{,}000}{3.51 + 7.18} = 13{,}900$ pounds per square inch. The actual unit stress in the member due to the same loads is equal to $\frac{58{,}700 + 60{,}000 + 35{,}700}{16.76} = 9{,}220$ pounds per square inch. Hence the efficiency is $\frac{13{,}900}{9{,}220} = 1.51$.

The upper and lower connections of the post are shown in Fig. 7. The rivets are all $\frac{3}{4}$-inch shop riveted. As the plates used are $\frac{3}{8}$ and $\frac{1}{2}$-inch thick, the shearing value of the rivets is the limiting condition. The shearing value of a $\frac{3}{4}$-inch rivet at 10,000 pounds per square inch is 4,420 pounds, but, as the wind load is considered, this may be increased by 25 percent. Therefore the number of rivets required is equal to $\frac{154{,}400}{1.25 \times 4{,}420} = 28$. There are just 28 rivets at the upper end which gives an efficiency of one but at the lower end there are but 24 connecting the angles of the

member and the stay plate, hence the efficiency is $\frac{24}{28} = 0.857$.

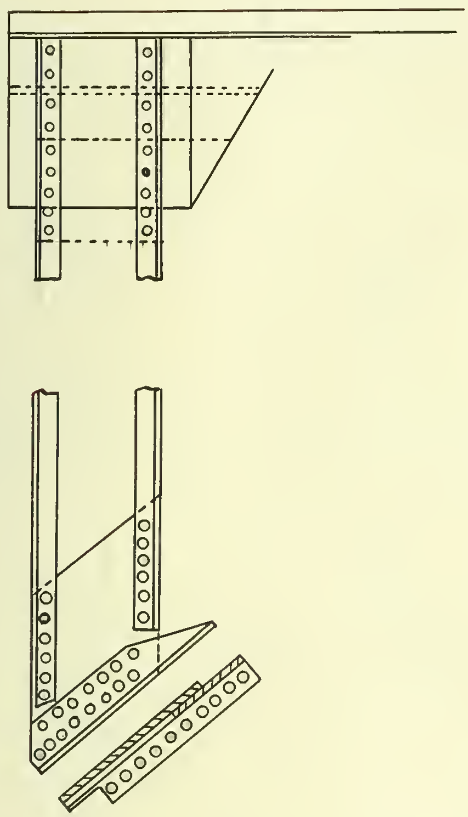


Fig. 7.

The rivets connecting the member to the inclined post are field rivets. As the height of the truss is 40 feet and the double panel is 39 feet long the proportion of the stress carried by these rivets is $\frac{154,400 \times 40}{\sqrt{39^2 + 40^2}} = 110,500$ pounds, and the number of rivets required is equal to $\frac{110,500}{1.25 \times \frac{2}{3} \times 4,420} = 30$. There are actually but 20 rivets and the efficiency is $\frac{20}{30} = 0.667$.

3(b). Post L_7U_7. End posts L_7U_7 and $L_7'U_7'$ are alike but the stresses in the former are somewhat the greater and hence it, only, will be investigated. The member is composed of four 6"x 6"x $\frac{9"}{16}$ angles, the section being shown by Fig. 8, and the stresses carried are 117,600 due to dead load, 120,100 due to live load, and 63,600 due to wind pressure.

The wind stress is $\frac{63,600}{117,600 + 120,100} \times 100 = 26.8$ percent of the combined dead and live load stresses and hence must be considered.

The total area of the member is $4 \times 6.43 = 25.72$ square inches. The moment of inertia about the axis a-a is

$$I = 4\left[22.07 + 6.43(14 - 1.71)^2\right] = 3,970 \text{ inches}^4$$

and the radius of gyration is, $r = \sqrt{\frac{3,970}{25.72}}$ = 12.44 inches. The moment of inertia about the axis b-b is

$$I = 4\left[22.07 + 6.43(10.5 + 1.71)^2\right] = 3,930 \text{ inches}^4$$

and the radius of gyration is, $r = \sqrt{\frac{3,930}{25.72}}$ = 12.13 inches. Therefore, the allowable dead load stress is equal to 22,000 - 80 x$\frac{480}{12.14}$ = 18,800 pounds per square inch, the length of the member being 49 x 12 = 480 inches, and the allowable live load stress is equal to $\frac{18,800}{2}$ = 9,400 poundsper square inch. The area required for dead load is $\frac{117,600}{18,800}$ = 6.26 square inches and the area required for live load is $\frac{120,100}{9,400}$ = 12.78 square inches.

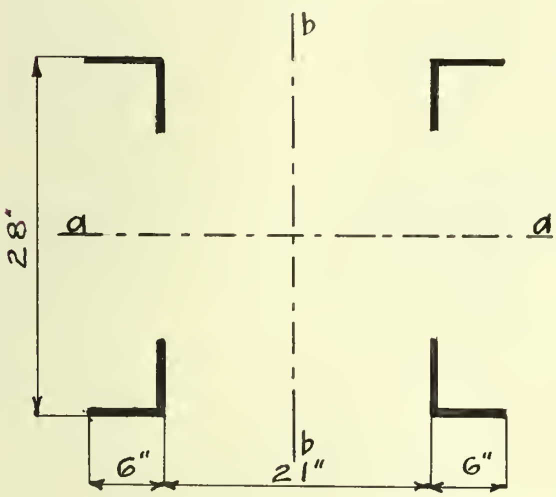


Fig. 8.

The average allowable unit stress in the member due to dead, live, and wind loads is $\frac{117,600 + 120,100}{6.26 + 12.76}$ =15,600 pounds per square inch. The actual unit stress due to the same loads is equal to $\frac{117,600 + 120,100 + 63,600}{25.72}$ = 11,700 pounds per square inch and hence the efficiency is $\frac{15,600}{11,700}$ = 1.33

The upper and lower connections of the post are shown in Fig.9. The rivets are $\frac{3}{4}$-inch shop. Since the thinest plate used is $\frac{3}{8}$-inch, the shearing value of the rivets is the limiting factor. The shearing value of a $\frac{3}{4}$-inch rivet is 1.25 x 4,420 = 5,525 pounds, and the required number of rivets is $\frac{301,300}{5,525}$ = 54.5. There are 64 rivets in the upper end, and 84 rivets in the lower end. The efficiencies are $\frac{64}{54.5}$ = 1.174, and $\frac{84}{54.5}$ = 1.54 respectively.

The pin at U_7 is $5\frac{3}{8}$ inches in diameter. As the allowable

bearing is 18,000 pounds per square inch the total thickness of plates required is, $t = \frac{301,300}{18\ 000 \times 5.375} = 3.11$ inches. As there are three $\frac{5}{8}$-inch plates, Fig. 10, on each side the total actual thickness is equal to $6 \times \frac{5}{8} = 3.75$ inches, giving an efficiency of $\frac{3.75}{3.11} = 1.205$. The stress carried by each set of plates is $\frac{301,300}{2} = 150,650$ pounds and $\frac{1}{3} \times 150,650 = 50,220$ pounds is taken by each plate.

The number of rivets required to connect the two pin plates to the stay plate is $\frac{2 \times 50,220}{5,525} = 18.2$, the actual number is 23 and hence the efficiency is $\frac{23}{18.2} = 1.264$.

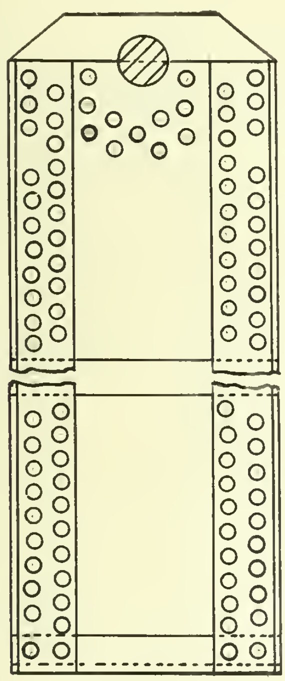


Fig. 9.

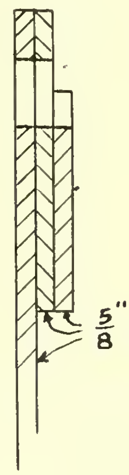


Fig. 10.

3(c). Inclined Post L_0U_1. The inclined end post L_0U_1 is composed of two angles $3'' \times 3'' \times \frac{3''}{8}$, two angles $3'' \times 4'' \times \frac{9''}{16}$, two web plates $16'' \times \frac{1''}{2}$ and one cover plate $18'' \times \frac{3''}{8}$. The section is shown by Fig.11.

The greatest stresses carried by the member are 140,700 pounds due to dead load, 143,400 pounds due to live load, and 114,400 pounds due to wind pressure.

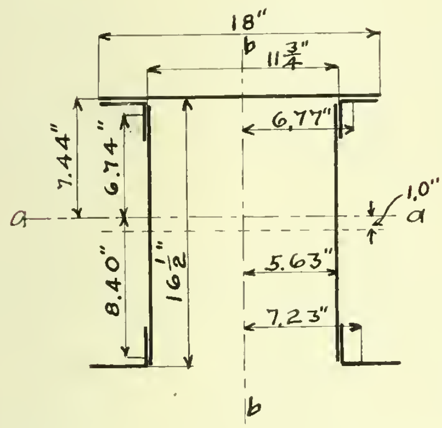


Fig. 11.

The area of the section is as follows

2- Ls- 3"x 3"x $\frac{3}{8}$" = 4.22 square inches

2 " 3"x 4"x$\frac{9}{16}$" = 7.24 " "

2 Web Pl.16"x$\frac{1}{2}$" = 16.00 " "

1 C. " 18"x$\frac{3}{8}$" = 6.75 " "

Total = 34.21 square inches.

It will be necessary to determine if there is eccentricity of longitudinal loading, and to do so the neutral axis of the section must be located. The static moment of the section about the top line of the cover plate as an axis is

Moment of cover plate	= 6.75 x 0.19	=	1.3 inches^3
" " 2- Ls- 3"x 3"x $\frac{3}{8}$"	= 4.22 x 1.27	=	5.4 "
" " 2 " 3"x 4"x $\frac{9}{16}$"	= 7.24 x 16.03	=	116.3 "
" " 2- Web Plates	= 16.00 x 8.63	=	138.4 "
Total		=	261.4 inches^3

The neutral axis is hence $\frac{261.4}{34.21}$ = 7.63 inches below the top of the section. Therefore the eccentricity is 8.63 - 7.63 = 1.0 inches, the center of the pins being at the center of the section.

The moment of inertia about the axis a-a is

Moment of angles	= $2(1.76 + 2.11 \times 6.74^2)$	=	195.5 inches^4
" " "	= $2(2.66 + 3.62 \times 8.40^2)$	=	517.5 "
" " Web Pl.	= $2(170.67 + 8.0 \times 1.0^2)$	=	357.0 "
" " C. "	= $0.8 + 6.75 \times 7.44^2$	=	374.1 "

Total Moment $I_{a-a} = 1,444.1$ inches^4

and the radius of gyration, $r_{a-a} = \sqrt{\frac{1,444.1}{34.21}} = 6.50$ inches.

The moment of inertia about the axis b-b is

Moment of angles	$= 2(1.76 + 2.11 \times 6.77^2)$	$=$	218.6 inches^4
" " "	$= 2(5.55 + 3.62 \times 7.23^2)$	$=$	390.3 "
" " Web Pl.	$= 2(0.17 + 8.0 \times 5.63^2)$	$=$	508.0 "
" " C. "	$=$		182.3 "
	I_{b-b}	$=$	1,299.2 inches^4

and the radius of gyration, $r_{b-b} = \sqrt{\frac{1,299.2}{34.21}} = 6.16$ inches.

The total length of the member is $\sqrt{40^2 + 39^2} = 55$ feet and 10 inches, but as the unsupported length is only one-half of this, or $l = \frac{55}{2} \times 12 + \frac{10}{2} = 335$ inches, the allowable unit stress for dead load is, $P = 22,000 - 80 \times \frac{335}{6.16} = 17,600$ pounds per square inch, and the allowable unit stress for live load is $\frac{17,600}{2} = 8,800$ pounds per square inch.

The required area for dead load stress is $\frac{140,700}{17,600} = 7.98$ square inches and the area required for live load stress is $\frac{143,400}{8,800} = 16.31$ square inches.

the average allowable unit stress for dead and live loads is $\frac{140,700 + 143,400}{7.98 + 16.31} = 11,700$ pounds per square inch.

The actual unit stress produced by these loads is

$$\frac{140,700 + 143,400}{34.21} = 8,300 \text{ pounds per square inch,}$$

and the unit stress produced by the wind is $\frac{114,400}{34.21} = 3,350$ pounds per square inch. The wind stress is $\frac{3,350}{8,300} \times 100 = 40.3$ percent of the average stress due to dead and live loads and must be considered.

The stress in the extreme fibre due to eccentric loading will be

$$f_e = \frac{M \times y}{I - \frac{P \times l^2}{10 \times E}}$$

where M is the moment produced by the axial stress, y is the distance from the neutral axis to the remote fibre, I is the moment of inertia, P is the axial stress, l is the unsupported length, and E is the modulus of elasticity of the material of which the average value of 28,000,000 will be used.

The stress in the upper fibre is

$$f_e = \frac{398{,}000 \times 1.0 \times 7.63}{1{,}444.1 - \frac{398{,}500 \times 335^2}{10 \times 28{,}000{,}000}} = + 2{,}365 \text{ pounds (tension)}$$

and the eccentric stress in the lower fibre is

$$f_e' = \frac{9.27}{7.63} \times 2{,}365 = - 2{,}870 \text{ pounds (compression).}$$

The stress due to the weight of the member will next be determined. the total weight of the member is as follows

2- 3" x 3" x $\frac{3"}{8}$ angles 50'- 1" long	=	783	pounds
2- 3" x 4" x $\frac{9"}{16}$ " 55'- 9" "	=	1,396	"
2- Web Plates 16" x $\frac{1}{2}$" x 56'- $\frac{3}{4}$" "	=	3,050	"
1- Cover " 18" x $\frac{3}{8}$" x 56'- $\frac{3}{4}$" "	=	1,287	"
Details and Lacing 23%	=	1,504	"
Total Weight	=	8,020	pounds.

The weight of the unsupported length is $\frac{8{,}020}{2}$ = 4,010 pounds and the bending moment due to the weight is, $M = \frac{1}{8} \cdot W \cdot l \cdot \cos \phi$ where ϕ is the angle the member makes with the horizontal. Therefore the the stress due to the weight in the upper fibre is

$$f_w = \frac{\frac{1}{8} \times 4{,}010 \times 335 \times \frac{39}{55.83} \times 7.63}{1{,}444.1 - \frac{398{,}500 \times 335^2}{10 \times 28{,}000{,}000}} = - 697 \text{ pounds}$$

(compression) and the stress due to weight in the lower fibre is

$$f_w' = \frac{9.25}{7.63} \times 697 = + 844 \text{ pounds (tension).}$$

The unit stress due to eccentric loading and the weight of the

member is, therefore, $\frac{2,870 - 844}{8,300} \times 100 = 24.4$ percent which is greater than the 10 percent allowed and must be considered.

Hence the actual unit stress in the member due to dead, live, and wind loads, eccentric load, and weight is greatest in the lower fibre and is, $P = 8,300 + 3,350 + 2,870 - 844 = 13,676$ pounds per square inch while the average allowable unit stress due to the same loadings is, $P = 11,700 + 0.25 \times 11,700 + 0.10 \times 11,700 = 15,800$ pounds per square inch and hence the efficiency is

$$E = \frac{15,800}{13,676} = 1.155.$$

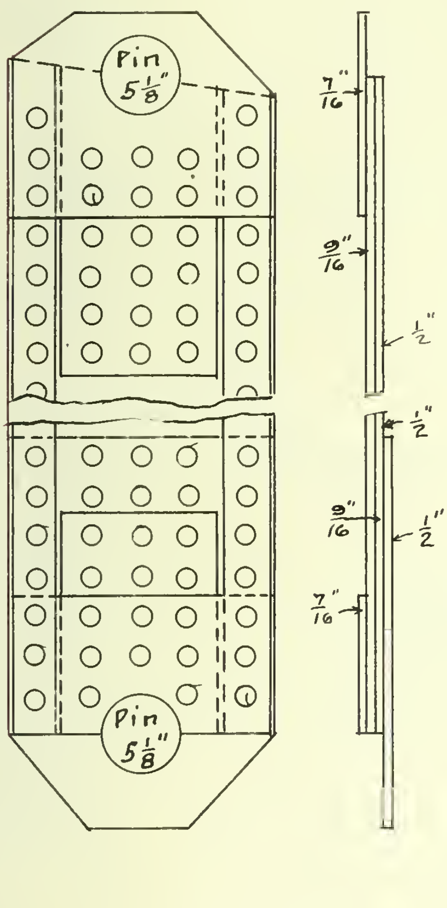


Fig. 12.

The upper and lower connections of the member are shown in Fig. 12. The rivets are all $\frac{3}{4}$-inch and are field rivets.

The maximum total stress to be transfered by the plates to the upper pin is 376,800 pounds, while that to be transfered to the lower pin is 398,500 pounds. Both pins are $5\frac{1}{8}$ inches in diameter, and hence the total thickness of plates required at the upper end is

$$t = \frac{376,800}{1.25 \times 18,000 \times 5.12} = 3.27$$

inches and at each side is $\frac{3.27}{2} = 1.63$ inches. The actual thickness of plates is, $t' = \frac{7}{16} + \frac{9}{16} + \frac{8}{16} = 1\frac{1}{2}$ inches. The efficiency of the plates is therefore $\frac{1.5}{1.63} = 0.92$.

The stress carried by the $\frac{7}{16}$-inch plate is equal to

$\frac{\frac{7}{16} \times \frac{376,800}{2}}{1.5}$ = 55,000 pounds, and as the shearing value of a $\frac{3}{4}$-inch rivet at 1.25 x 10,000 pounds per square inch is 1.25 x 4,420 = 5,525 pounds, the required number of rivets for this plate is $\frac{55,000}{5,525}$ = 9.5, while the actual number is 12 giving an efficiency of $\frac{12}{9.5}$ = 1.265.

The proportion of stress carried by the $\frac{9}{16}$- inch plate is $\frac{\frac{9}{16} \times 55,000}{\frac{7}{16}}$ = 70,700, and the required no of rivets is $\frac{70,700}{5,525}$=12.8. The total required number of rivets to connect the pin plates to the web plate therefore equal to 9.5 + 12.8 = 22.3 while the actual number is 24 giveing an efficiency of $\frac{24}{22.3}$ = 1.075.

At the lower end, the required thickness of plates is,

$t = \frac{398,500}{1.25 \times 18,000 \times 5.12}$ = 3.46 inches which is $\frac{3.46}{2}$ = 1.73 inches at each side. The actual thickness is $\frac{7}{16} + \frac{9}{16} + \frac{1}{2} + \frac{1}{2}$ = 2.0 inches giveing an efficiency of $\frac{2.0}{1.73}$ = 1.154.

The proportion of stress carried by the $\frac{7}{16}$-inch plate is $\frac{\frac{7}{16} \times \frac{398,500}{2}}{2}$ = 43,600 and the required number of rivets is $\frac{43,600}{5,525}$ = 7.88 while the actual number is 14. The proportion of stress carried by the $\frac{9}{16}$-inch plate is $\frac{9}{16} \times \frac{16}{7} \times$ 43,600 pounds, and the required number of rivets is $\frac{56,000}{5,525}$ = 10.14. For the two plates the required number of rivets is 10.14 + 7.88 = 18.02 while the actual number is 20 giving an efficiency of $\frac{20}{18.02}$ = 1.11.

The proportion of stress carried by the $\frac{1}{2}$-inch plate is $\frac{\frac{1}{2} \times 43,600}{\frac{7}{16}}$ = 49,800 pounds and the required number of rivets is $\frac{49,800}{5,525}$ = 9 while the actual number is 34.

The allowable bearing in the half-inch web plate is

$1.25 \times 18{,}000 \times \frac{3}{4} \times \frac{1}{2} = 8{,}450$ pounds per rivet which is less than the value of the same rivet in single shear. The required number of rivets for bearing in the web plate is $\frac{43{,}600 + 56{,}000 + 49{,}800}{8{,}450}$ $= 17.67$ while the actual number is 34, the efficiency being $\frac{34}{17.67} = 1.925$.

The distance between the gage lines of the upper angles is $15\frac{1}{4}$ inches, and hence the thickness of the cover plate should be at least, $t = \frac{15.25}{40} = \frac{3}{8}$-inch which is the thickness of the plate used.

3(d). Inclined Post $L_0'U_1$ This post is the same as L_0U_1 in all respects except that $\frac{7}{16}$-inch web plates are used in place of the $\frac{1}{2}$-inch plates of L_0U_1 The greatest stresses carried by the member are 133,300 pounds due to dead load, 139,000 pounds due to live load, and 114,400 pounds due to wind pressure.

The area of the section is as follows

Area 2 angles	$3" \times 3" \times \frac{3"}{8}$	=	4.22	square inches
" 2 "	$3" \times 4" \times \frac{9"}{16}$	=	7.24	" "
" 2 web plates	$16" \times \frac{7"}{16}$	=	14.00	" "
" 1 cover "	$18" \times \frac{3"}{8}$	=	6.75	" "
	Total area	=	32.21	square inches.

The static moment of the section about the top fibre as an axis is

Moment of cover plate	$18" \times \frac{3"}{8}$	=	6.75×0.19	= 1.3	inches^3
" " 2 web "	$16" \times \frac{7"}{16}$	=	14.00×8.6	=120.8	"
" " 2 angles	$3" \times 3" \times \frac{3"}{8}$	=	4.22×1.3	= 5.4	"
" " 2 "	$3" \times 4" \times \frac{9"}{16}$	=	7.24×16.0	=116.3	"
	Total Moment			= 243.8	inches^3

The neutral axis is hence $\frac{243.8}{34.21} = 7.13$ inches below the top of

the section. Therefore the eccentricity is $8.63 - 7.13 = 1.5$ inches.

The moment of inertia about the axis a-a, Fig. 13, is

Moment of angles	$= 2(1.76 + 2.11 \times 6.24^2) =$	168.0	inches^4
" " "	$= 2(2.66 + 3.62 \times 8.90^2) =$	580.1	"
" " Web Pl.	$= 2(149.33 + 7.0 \times 1.5^2) =$	330.2	"
" " C. "	$= 6.75 \times 6.94^2 =$	326.5	"
	$I_{a-a} =$	1,404.8	inches^4,

and the radius of gyration, $r_{a-a} = \sqrt{\dfrac{1,404.8}{32.21}} = 6.61$ inches.

The moment of inertia about the axis b-b is

Moment of angles	$= 2(1.76 + 2.11 \times 6.77^2) =$	218.6	inches^4
" " "	$= 2(5.55 + 3.62 \times 7.23^2) =$	390.3	"
" " Web Pl.	$= 2(0.11 + 7.0 \times 5.66^2) =$	450.2	"
" " C. "	$=$	182.3	"
	$I_{b-b} =$	1,241.4	inches^4,

and the radius of gyration, $r_{b-b} = \sqrt{\dfrac{1,241.4}{32.21}} = 6.21$ inches.

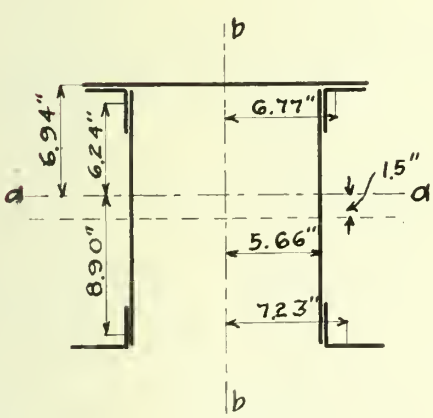


Fig. 13.

The allowable unit stress for dead load is

$$P = 22,000 - 80 \times \frac{335}{6.21} = 17,680$$

pounds per square inch and the allowable unit stress for live load is $\dfrac{17,860}{2} = 8,840$ pounds per square inch.

The required area for dead load stress is $\dfrac{133,300}{17,680} = 7.54$ square inches, and the area required live load is $\dfrac{139,000}{8,840} = 15.74$ square inches.

The average allowable unit stress for dead and live loads is $\frac{133,300 + 139,000}{7.54 + 15.74}$ = 11,700 pounds per square inch.

The actual unit stress produced by these loads is $\frac{133,300 + 139,000}{31.21}$ = 8,460 pounds per square inch and the unit stress produced by the wind is $\frac{114,400}{32.21}$ = 3,550 pounds per square inch.

The stress in the extreme lower fibre of the member due to eccentric loading is

$$f'_e = \frac{386,700 \times 1.5 \times 9.75}{1,404.8 - \frac{386,700 \times 335^2}{10 \times 28,000,000}} = -4\ 520 \text{ pounds (com-}$$

pression) per square inch.

The weight of the member is as follows

2- 3"x 3"x $\frac{3"}{8}$ angles	50'- 1"	long	= 783	pounds
2- 3"x 4"x $\frac{9"}{16}$ "	55'- 9"	"	= 1,396	"
2- 16"x $\frac{7"}{16}$ web plates	56'- 1"	"	= 2,620	"
1- 18"x $\frac{3"}{8}$ cover "	56'- 1"	"	= 1,287	"
Details and lacing 25%			= 1,504	"
	Total Weight		= 7,590	pounds.

The weight of the unsupported length is $W = \frac{7,590}{2}$ = 3,795 pounds, and the stress in the extreme lower fibre, as before, is

$$f_w = \frac{\frac{1}{8} \times 3,795 \times 335 \times \frac{39}{55.83} \times 9.75}{1,404.8 - \frac{386,700 \times 335^2}{10 \times 28,000,000}} = +\ 870 \text{ pounds}$$

(tension) per square inch.

The actual unit stress in the member due to dead, live, and wind loads, eccentric load, and weight of member is greatest in the lower fibre and is, P = 8,460 + 3,550 + 4,520 - 870 = 15,660 pounds per square inch, while the allowable unit stress due to the same loadings is, P = 11,700 + 0.25 x 11,700 + 0.10 x 11,700 = 15,800

pounds per square inch and hence the efficiency is $\frac{15,800}{15,660} = 1.01$.

The upper and lower connections are the same as the respective connections of the post L_0U_1.

The total thickness of plates required at the upper end is,

$t = \frac{364,700}{1.25 \times 18,000 \times 5.12} = 3.16$ inches which amounts to $\frac{3.16}{2} = 1.58$ inches on each side while the actual thickness is, $t' = \frac{7''}{16} + \frac{9''}{16} + \frac{7''}{16} = 1\frac{7}{16}$ inches. The efficiency is hence $\frac{1\ 7/16}{1.58} = 0.91$.

At the lower end the required thickness is $\frac{386,700}{1.25 \times 18,000 \times 5.12} = 3.36$ inches, or $\frac{3.36}{2} = 1.68$ inches on each side. The actual thickness is, $t' = \frac{7}{16} + \frac{9}{16} + \frac{1}{2} + \frac{7}{16} = 1\frac{15}{16}$ inches giving an efficiency of $\frac{1\frac{15}{16}}{1.68} = 1.153$.

As the riveting was ample for post L_0U_1 it will be considered to be so for post $L_0'U_1'$.

4(a). Inclined Struts. Posts L_2U_3, U_3L_4, $L_2'U_3'$, and $U_3'L_4'$. These posts are all alike, each one being composed of four angles $3'' \times 5'' \times \frac{7''}{16}$, the section being as shown in Fig. 14. The greatest compressive stress occurs in member L_2U_3 and are 46,800 pounds due to dead load and 64,800 pounds due to live load. However, in U_3L_4 there is a reversal of stress, the stress due to dead load is 11,700 pounds (tension); and the stresses due to live load are 47,800 pounds (tension) and 35,800 pounds (compression). It will be necessary to investigate for the maximum of each kind of stress.

The area of the section is equal to $4 \times 3.31 = 13.24$ square inches and the moment of inertia about the axis a-a is

$$I_{a-a} = 4\left[2.32 + 3.31(8 - 0.73)^2\right] = 712 \text{ inches}^4,$$

and the radius of gyration about the axis a-a is $\sqrt{\frac{712}{13.25}} = 7.32$ inches.

The moment of inertia about the axis b-b is

$$I_{b-b} = 4\left[8.43 + 3.31(5.5 + 1.73)^2\right] = 727 \text{ inches}^4,$$

and the radius of gyration is $\sqrt{\frac{727}{13.24}} = 7.41$ inches.

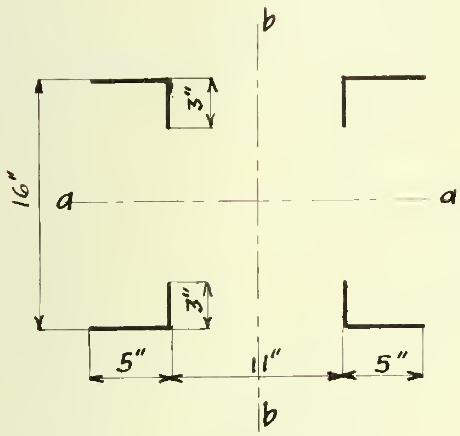


Fig. 14.

The unsupported length of the is the same as for member L_0U_1, i.e. 335 inches, and hence the allowable compressive stress is $22,000 - 80 \times \frac{335}{7.32} = 18,340$ pounds per square inch for dead load and $\frac{18,340}{2} = 9,170$ pounds per square inch for live load.

Because of the reversal of stress in member U_3L_4 each kind must be increased by $\frac{8}{10}$ of the smaller, but when the compressive stress is so increased it will not be as large as the actual compressive stress in member L_2U_3. In the later member the area required for dead load stress is $\frac{46,800}{18,340} = 2.55$ square inches, and the area required for live load stress is $\frac{64,800}{9,170} = 7.07$ square inches. The average allowable stress due to these loads is $\frac{46,800 - 64,800}{2.55 - 7.07}$ = 11,600 pounds per square inch.

The weight of the member is

2- angles 3"x 5"x $\frac{7"}{16}$ x 53'- 11" long	=	1,210 pounds
2 " " " " x 54'- 4" "	=	1,215 "
Details and lacing	=	1,230 "
Total Weight	=	3,655 pounds,

and for the unsupported length the weight is $\frac{3,655}{2} = 1,828$ pounds. By the formula on p. 29, the stress in the extreme fibre, due to the weight of the member, is

$$f_w = \frac{\frac{1}{8} \times 1{,}828 \times 335 \times \frac{39}{55.83} \times 8}{712 - \frac{116{,}600 \times 335^2}{10 \times 28{,}000{,}000}} = \pm 640 \text{ pounds per}$$

square inch, which is $\frac{640}{11,600} \times 100 = 5.52$ percent of the average allowable unit stress due to dead and live load. This being less than the 10 percent allowed, the stress due to the weight of the member is not considered.

Hence the required area is $2.55 + 7.07 = 9.62$ square inches while the actual area as stated on p. 35 is 13.24 square inches giving an efficiency of $\frac{13.24}{9.62} = 1.375$.

The tensile stresses to be considered are 11,700 due to dead load and $47,800 + \frac{8}{10} \times 35,800 = 76,500$ due to live load.

The allowable tensile stress is 25,000 pounds per square inch for dead load and 12,500 pounds per square inch for live load. The required dead load area is, therefore, $\frac{11,700}{25,000} = 0.47$ square inches, while the area required for live load is $\frac{76,500}{12,500} = 6.13$ square inches, giving a total required area of $0.47 + 6.13 = 6.6$ square inches.

As the angles are connected to the gusset plates at the ends by the 3-inch legs only these legs only can be considered as effective section according to Article 46 of the Specifications. Since the rivets are $\frac{3}{4}$-inch in diameter, the diameter of the rivet holes must be assumed as $\frac{7}{8}$-inch. The effective net area of the section is hence equal to $4(3 - \frac{7}{8}) \times \frac{7}{16} = 3.72$ square inches, giving an efficiency of $\frac{3.72}{6.6} = 0.564$. The member is field rivited to the gusset plate at each of the ends, the rivets used being $\frac{3}{4}$-inch. The gusset plates are $\frac{3}{8}$-inch thick, and hence the shear on the rivets is less than the bearing value in the plate. The greatest stress to be transfered to the plates is 111,600 pounds and the

number of rivets required is $\frac{111,600}{\frac{2}{3} \times 4,420}$ = 38, while the actual number is but 28 giving a efficiency of $\frac{28}{38}$ = 0.737.

4(b). Inclined Struts U_5L_6 and $U'_5L'_6$. These members are alike, being composed of four angles 4" x 5" x $\frac{5}{8}$" in cross section. The section of the member is shown by Fig. 15.

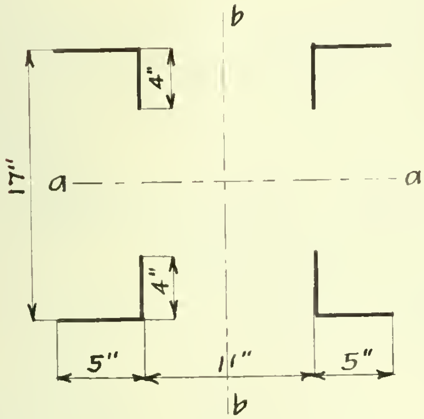


Fig. 15.

The stresses in member $U'_5L'_6$ are somewhat the larger. They are 98,400 pounds due to dead load and 105,600 pounds due to live load. The total area of the member is 4 x 5.23 = 20.92 square inches and the moment of inertia of the section about axis a-a is

$$I_{a-a} = 4\left[7.14 + 5.23(8.5 - 1.12)^2\right] = 1,174 \text{ inches}^4,$$

and the radius of gyration about the same axis is, $r_{a-a} = \sqrt{\frac{1,174}{20.92}}$ = 7.64 inches.

The moment of inertia about the axis b-b is

$$I_{b-b} = 4\left[12.62 + 5.23(5.5 + 1.62)^2\right] = 1,114 \text{ inches}^4,$$

and the radius of gyration is $\sqrt{\frac{1,114}{20.92}}$ = 7.44 inches.

The allowable dead load stress is, therefore,

$$P = 22,000 - 80 \times \frac{335}{7.44} = 18,400 \text{pounds per square inch,}$$

as the unsupported length is the same as for the other inclined struts, that is 335 inches, and the allowable live load stress is $\frac{18,400}{2}$ = 9,200 pounds per square inch. Hence, the required dead load area is $\frac{98,400}{18,400}$ = 5.35 square inches and for live load is $\frac{105,600}{9,200}$ = 11.47 square inches. This gives an average allowable unit stress

of $\frac{98,400 + 105,600}{3.35 + 11.47}$ = 12,100 pounds per square inch.

The weight of the member is

2- angles 4"x 5"x $\frac{5}{8}$" x 54'- 3" long	= 1,954	pounds
2 " " " " 54'- 6" "	= 1,960	"
Details and Lacing	= 2,450	"
Total Weight	= 6,364	pounds,

and for the unsupported length the weight is, $W = \frac{6,364}{2}$ = 3,182 pounds.

The stress due to the weight of the member is

$$f_w = \frac{\frac{1}{8} \times 3\,182 \times 335 \times \frac{39}{59.83} \times 8.5}{1,174 - \frac{204,000 \times 335^2}{10 \times 28,000,000}} = \pm\ 687 \text{ pounds per}$$

square inch, and as this is but $\frac{687}{12,100} \times 100 = 5.68$ percent of the average allowable stress it is not considered.

The required area is therefore 5.35 + 11.47 = 16.82 square inches while the actual area is 20.92 square inches, giving an efficiency of $\frac{20.92}{16.82} = 1.19$.

The connections of the member are the same as for the similar members, L_2U_3 and $L'_2U'_3$, except that the lower end is shop rivited.

The greatest stress to be transfered to the upper gusset plate is 185,100 pounds, and the required number of rivets is $\frac{185,100}{\frac{2}{3} \times 4,420}$ = 63, while the actual number is 44 giving an efficiency of $\frac{44}{63} = 0.70$.

The greatest stress to be transfered to the lower gusset plate is 204,000 pounds, and the required number of rivets is $\frac{204,000}{4,420}$ = 46.2, while the actual number is 44 giving an efficiency of $\frac{44}{46.2} = 0.952$.

5. Upper Chord. (a). Section U_0U_1. This section of the upper chord is composed of 4 angles $2\frac{1}{2}" \times 2\frac{1}{2}" \times \frac{1}{4}"$ in spite of the fact that Art. 97 of the Specifications states that for main members and their connections no material shall be used of a less thickness than $\frac{5}{16}$ of an inch. The stresses carried by the member are 8,200 pounds due to dead load, 8,400 pounds due to live load, and 2,900 pounds due to wind pressure. This wind stress is $\frac{2,900}{16,600} \times 100 = 17.45$ percent of the combined dead and live load stresses, and since this is less than the 25 percent required by the Specifications it is not to be considered.

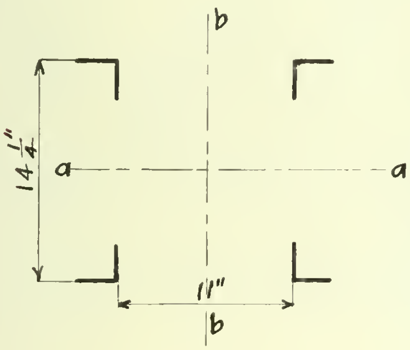


Fig. 16.

The section is shown in Fig.16. The moment of inertia about axis a-a is,

$$I_{a-a} = 4\left[0.70 + 1.19(7.13 - 0.72)^2\right]$$

$I_{a-a} = 199.2$ inches^4, and as the total area of the section is $4 \times 1.19 = 4.76$ square inches, the radius of gyration $r_{a-a} = \sqrt{\frac{199.2}{4.76}} = 6.46$ inches.

The moment of inertia about the axis b-b is, $I_{b-b} = 4\left[0.70 + 1.19(5.5 + 0.72)^2\right] = 18.74$ inches^4, and the radius of gyration is $\sqrt{\frac{187.4}{4.76}} = 6.19$ inches.

The allowable stress in the member is

$$P = 22,000 - 80 \times \frac{234}{6.19} = 19,000 \text{ pounds per square inch}$$

for dead load, while for live load it is $\frac{19,000}{2} = 9,500$ pounds per square inch. This requires an area of $\frac{8,200}{19,000} = 0.43$ square inches for dead load, and $\frac{8,400}{9,500} = 0.89$ square inches for live load which gives an average allowable stress of $\frac{8,200 + 8,400}{0.43 + 0.89} = 12,600$ pounds per square inch.

The weight of the member is

4- angles $2\frac{1}{2}$" x $2\frac{1}{2}$" x $\frac{1}{4}$" x 19'- 6" long = 320 pounds

Details and Lacing 25% = 80 "

Total Weight = 400 pounds,

and the stress in the remote fibre due to weight is

$$f_w = \frac{\frac{1}{8} \times 400 \times 234 \times 7.13}{199.2 - \frac{16,600 \times 234^2}{10 \times 28,000,000}} = \pm 425 \text{ pounds per square inch,}$$

since this is only $\frac{425}{12,600} \times 100 = 3.37$ percent of the average allowable stress it is not considered.

Therefore the required area of the member is 0.43 + 0.89= 1.32 square inches while the actual area is 4.76 square inches giving an efficiency of $\frac{4.76}{1.32} = 3.61$.

5(b). Section U_1 to U_3. The section of the upper chord from U_1 to U_3 is composed of 2 angles 3"x 3"x $\frac{5"}{16}$, 2 angles 3"x 4"x $\frac{7"}{16}$, 2 web plates 14"x $\frac{3"}{8}$, and 1 cover plate 18"x $\frac{3"}{8}$. The section is shown by Fig. 17. The greatest stresses carried by the member are 172,200 pounds due to dead load, 176,000 pounds due to live load, and 91,500 pounds due to wind pressure.

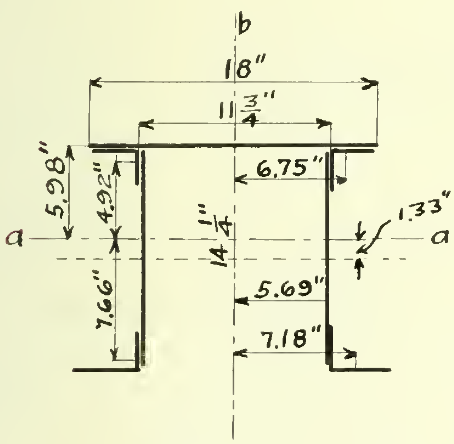


Fig. 17.

The area of the section is as follows

2- Ls- 3"x 3"x $\frac{5"}{16}$	= 3.56	square	inches
2 " 3"x 4"x $\frac{7"}{16}$	= 5.74	"	"
2 Web Pl. 14"x $\frac{3"}{8}$	= 10.50	"	"
1 C. " 18"x $\frac{3"}{8}$	= 6.75	"	"
Total	= 26.55	square	inches

The static moment of the section about the top line of the cover plate as an axis is

Moment of Cover plate $= 6.75 \times 0.19 = 1.3$ inches^3

" " 2 Web " $= 10.5 \times 7.50 = 78.7$ "

" " 2 s $3" \times 3" \times \frac{5"}{16} = 3.56 \times 1.25 = 4.5$ "

" " 2 " $3" \times 4" \times \frac{7"}{16} = 5.74 \times 13.03 = 79.4$ "

Total $= 163.9$ inches^3.

The neutral axis is hence $\frac{163.9}{26.55} = 6.17$ inches below the top of the section. Therefore the eccentricity is $7.50 - 6.17 = 1.33$ inches, the center of the pins being at the center of the section.

The moment of inertia about the axis a-a is,

Moment of angles $= 2(1.51 + 1.78 \times 4.92^2) = 89.2$ inches^4

" " " $= 2(2.18 + 2.87 \times 7.66^2) = 342.4$ "

" " web pl. $= 2(85.8 + 5.25 \times 1.33^2) = 190.1$ "

" " cov. " $= 0.08 + 6.75 \times 5.98^2) = \underline{241.8}$ "

$I_{a-a} = 863.5$ inches^4,

and the radius of gyration, $r_{a-a} = \sqrt{\frac{863.5}{26.55}} = 5.70$ inches.

The moment of inertia about axis b-b is,

Moment of angles $= 2(1.76 + 1.78 \times 6.75^2) = 166.2$ inches^4

" " " $= 2(4.52 + 2.87 \times 7.18^2) = 300.6$ "

" " web pl. $= 2(0.07 + 5.25 \times 5.69^2) = 341.2$ "

" " cov. " $= \underline{182.3}$ "

$I_{b-b} = 990.3$ inches^4,

and the radius of gyration, $r_{b-b} = \sqrt{\frac{990.3}{26.55}} = 6.22$ inches.

The unsupported length of the member is $19 \times 12 + 6 = 234$ inches and therefore the allowable stress is $22,000 - 80 \times \frac{234}{5.70}$ $= 18,700$ pounds per square inch for dead load, and for live load the allowable stress is $\frac{18,700}{2} = 9,350$ pounds per square inch.

The area for dead load is $\frac{172,200}{18,700} = 9.20$ square inches and the area required for live load is $\frac{176,000}{9,350} = 18.82$ square inches.

The average allowable unit stress for dead and live loads is $\frac{172,200 + 176,000}{9.20 + 18.82}$ = 12,400 pounds per square inch. The actual unit stress produced by these loads is $\frac{172,200 + 176,000}{26.55}$ = 13,100 pounds per square inch and the unit stress produced by the wind is $\frac{91,500}{26.55}$ = 3,440 pounds per square inch. The wind stress is $\frac{3,440}{13,100}$ x 100 = 26.3 per cent of the average stress due to dead and live loads and must be considered.

The stress in the upper fibre due to eccentric loading is

$$fe = \frac{439,700 \times 1.33 \times 6.17}{863.5 - \frac{439,700 \times 234^2}{10 \times 28,000,000}} = +4,640$$ pounds (tension) per square inch and the eccentric stress in the lower fibre is

$$f'e = \frac{8.46}{6.17} \times 4,640 = -6,370$$ pounds (compression) per square inch.

The weight of the unsupported length of the member is

2 - 3"	3"	$\frac{5''}{16}$	angles	19'-6"	long	= 238	pounds
2 - 3"	4"	$\frac{7''}{16}$	"	19'-6"	"	= 382	"
2 - Web plates	14"	$\frac{3''}{8}$		19'-6"	"	= 696	"
1 - Cover "	18"	$\frac{3''}{8}$		19'-6"	"	= 448	"
Details and Lacing - 25%						= 441	"
				Total Weight		= 2,205	pounds.

The stress in the upper fibre due to the weight of the member is

$$fw = \frac{\frac{1}{8} \times 2\,205 \times 234 \times 6.17}{863.5 - \frac{439,700\ \ 234}{10 \times 28,000,000}} = -512$$ pounds (compression),

and the stress in the lower fibre due to weight is $f'w = \frac{8.46}{6.17}$ 512 = +700 pounds (tension).

The unit stress due to eccentric loading and the weight of the member is, therefore, $\frac{6,370 - 700}{13,100} \times 100$ = 43.2 per cent and must be considered.

Hence the actual unit stress in the member due to dead, live, and wind loads, eccentric load, and weight is

P = 13,100 + 3,440 + 6 370 + 700 = 22,210 pounds per square inch, while the average allowable unit stress due to the same loads is 12,400 + 0.25 x 12,400 + 0.10 x 12,400 = 16,740 pounds per square inch and hence the efficiency is $\frac{16,740}{22,210}$ = 0.753.

5(c). Section U_3U_5. The section of the upper chord from U_3 to U_5 is composed of two angles 3"x 3"x $\frac{3}{8}$", two angles 3"x 4"x $\frac{1}{2}$", two web plates 14"x $\frac{1}{2}$", and one cover plate 18"x $\frac{3}{8}$". The section is shown by Fig. 18. The greatest stresses carried by the member are 204,800 pounds due to dead load, 209,400 pounds due to live load, and 103,000 pounds due to the pressure of the wind.

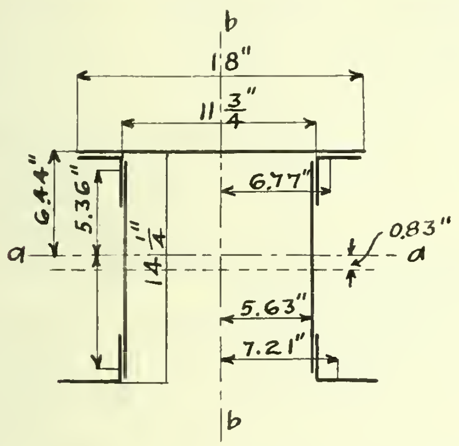


Fig. 18.

The area of the section is as follows,

2- Ls-3"x 3"x $\frac{3}{8}$"	=	4.22	square in.
2 " 3"x 4"x $\frac{1}{2}$"	=	7.50	" "
2 Web Pl. 14"x$\frac{1}{2}$"	=	14.00	" "
1 C. " 18"x $\frac{3}{8}$"	=	6.75	" "
Total Area	=	32.47	square in.

The static moment of the section about the top line of the cover plate is,

Moment of cover plate	= 6.75 x 0.19	=	1.3 inches^3
" " 2 web "	= 14.00 x 7.50	=	105.0 "
" " 2- Ls-3"x 3"x $\frac{3}{8}$"	= 4.22 x 1.27	=	5.4 "
" " 2- " 3"x 4"x $\frac{1}{2}$"	= 7.50 x 13.8	=	103.6 "
Total		=	215.3 inches^3.

The neutral axis is therefore $\frac{215.3}{32.47}$ = 6.63 inches below the top of the section and this gives an eccentricity of 7.50 - 6.63 = 0.87 inches.

The moment of inertia about the axis a-a is

Moment of angles	= $2(1.76 + 2.11 \times 5.36^2)$	= 125.2	inches^4
" " "	= $2(2.42 + 3.75 \times 7.17^2)$	= 338.8	"
" " web pl.	= $2(114.3 + 7.0 \times 0.83^2)$	= 238.3	"
" " cov. "	= $0.08 + 6.75 \times 6.44^2$	= 280.2	"
	I_{a-a}	= 982.5	inches^4,

and the radius of gyration, $r_{a-a} = \sqrt{\frac{982.5}{32.47}} = 5.50$ inches.

By comparing this section and results with those on p. 40 we see that this will be the least radius of gyration and it will be unnecessary to compute the moment of inertia about axis b-b.

As the unsupported length of the member is 234 inches, the allowable stresses are, for dead load $P = 22,000 - 80 \times \frac{234}{5.50} = 18,600$ pounds per square inch, and for live load $P = \frac{18,600}{2} = 9,300$ pounds per square inch.

The required area for dead load is $\frac{204,800}{18,600} = 11.06$ square inches, and for live load $\frac{209,400}{9,300} = 22.44$ square inches. The average allowable stress for dead and live load is $\frac{204,800 + 209,400}{11.06 + 22.44} =$ 12,350 pounds per square inch. The actual unit stress produced by these loads is $\frac{204,800 + 209,400}{32.47} = 12,740$ pounds per square inch, and the unit stress produced by the wind is $\frac{103,000}{32.47} = 3,170$ pounds per square inch. The wind stress is $\frac{3,170}{12,740} \times 100 = 24.8$ percent of the average stress due to dead and live loads, and as this is less than the 25 percent allowable it is not necessary to consider it.

The stress in the lower fibre due to eccentricity of longitudinal loading is

$$f_e = \frac{414,200 \times 0.83 \times 8.00}{982.5 - \frac{414,200 \times 234^2}{10 \times 28,000,000}} = -\ 3,050 \text{ pounds (compression).}$$

The weight of the unsupported length of the member is

2- 3"x3"x $\frac{3''}{8}$ angles = 282 pounds

2- 3"x 4"x $\frac{1''}{2}$ " = 432 "

2- Web Plates 14"x$\frac{1}{2}$" = 928 "

1- Cover " 18"x $\frac{3''}{8}$ = 448 "

Details and Lacing = 512 "

Total Weight = 2,602 pounds.

The stress in the lower fibre due to the weight of the member is

$$f_w = \frac{\frac{1}{8} \times 2,602 \times 234 \times 8.00}{982.5 - \frac{414,200 \times 234^2}{10 \times 28,000,000}} = + 680 \text{ pounds (tension).}$$

The unit stress due to eccentric loading and the weight of the member is, therefore, $\frac{3,050 - 680}{12,740} \times 100 = 18.6$ percent and must be considered.

Hence the actual unit stress in the member due to dead and live loads, eccentric loading, and weight, is

$$P = 12,740 + 3,050 - 680 = 15,100 \text{ pounds per square inch}$$

while the average allowable unit stress due to the same loads is 12,350 + 0.10 x 12,350 = 13,600 pounds per square inch, and hence the efficiency is $\frac{13,600}{15,100} = 0.90$.

5(d). Section U_5U_7. The section of the upper chord from U_5 to U_7 is composed of two angles 3"x 3"x $\frac{5''}{16}$, two angles 3"x 4"x $\frac{5''}{16}$, and two web plates 14"x $\frac{5''}{16}$. The section is shown by Fig. 19. The greatest stresses carried by the member are 106,400 pounds due to dead load, 108,800 pounds due to live load, and 100,100 pounds due to wind pressure.

The area of the section is as follows,

2- Ls 3"x 3"x $\frac{5''}{16}$ = 3.56 square inches

2- " 3"x 4"x $\frac{5''}{16}$ = 4.18 " "

2 Web Pl. 14"x $\frac{5''}{16}$ = 8.75 " "

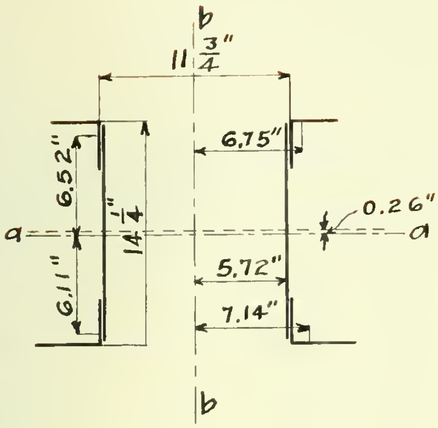


Fig. 19.

Total Area = 16.49 square inches.

The static moment about the top of the section is

2- Web Pl.	= 8.75 x 7.13=	62.4
2- Ls-3"x3"x$\frac{5''}{16}$	= 3.56 x 0.87=	3.1
2- " 3"x4"x$\frac{5''}{16}$	= 4.18 x 13.5=	56.4
Total		= 121.9

The neutral axis is, therefore, $\frac{121.9}{16.49}$ = 7.39 inches below the top of the section, or 7.39 - 7.13 = 0.26 inches below the center of the section, and hence the eccentricity is 0.26 inches.

The moment of inertia about the axis a-a is,

Moment of angles $= 2(1.51 + 1.78 \times 6.52^2) = 154.6$ inches^4

" " " $= 2(1.65 + 2.09 \times 6.11^2) = 159.8$ "

" " web pl. $= 2(71.47 + 4.38 \times 0.26^2) = \underline{143.5}$ "

$I_{a-a} = 457.9$ inches^4,

and the radius of gyration is, $r_{a-a} = \sqrt{\frac{457.9}{16.49}} = 5.28$ inches.

The moment of inertia about the axis b-b is,

Moment of angles $= 2(1.51 + 1.78 \times 6.75^2) = 165.8$ inches^4

" " " $= 2(2.38 + 2.09 \times 7.14^2) = 220.4$ "

" " web pl. $= 2(0.05 + 4.38 \times 5.72^2) = \underline{286.9}$ "

$I_{b-b} = 673.1$ inches^4,

and the radius of gyration is, $r_{b-b} = \sqrt{\frac{673.1}{16.49}} = 6.38$ inches.

The unsupported length being 234 inches, the allowable dead load stress is $22,000 - 80 \times \frac{234}{5.28} = 18,480$ pounds per square inch; and the allowable live load stress is $\frac{18,480}{2} = 9,240$ pounds per square inch.

The required area for dead load is $\frac{106,400}{18,480} = 5.76$ square inches, and the required live load area is $\frac{108,800}{9,240} = 11.76$ square inches.

The average allowable unit stress for dead and live loads is $\frac{106,400 + 108,800}{5.76 + 11.76} = 12,270$ pounds per square inch. The actual unit stress produced by the same loads is $\frac{106,400 + 108,800}{16.49} = 13,040$ pounds per square inch, and the wind load stress is $\frac{100,100}{16.49} = 6,070$ pounds per square inch. The wind stress is $\frac{6,070}{13,040} \times 100 = 46.5$ percent of the average allowable stress due to dead and live load and must be considered.

The stress in the upper fibre due to eccentric loading is

$$f_e = \frac{315,300 \times 0.26 \times 7.39}{457.9 - \frac{315,300 \times 234^2}{10 \times 28,000,000}} = -\ 1,530 \text{ pounds (compression).}$$

The weight of the unsupported length, 19 feet and 6 inches, of the member is

2- 3"x 3"x $\frac{5"}{16}$ angles	=	238	pounds
2- 3"x 4"x $\frac{5"}{16}$ "	=	281	"
2- Web Plates 14"x $\frac{5"}{16}$	=	580	"
Details and Lacing	=	275	"
Total Weight	=	1,374	pounds.

The stress in the upper fibre due to the weight of the member is

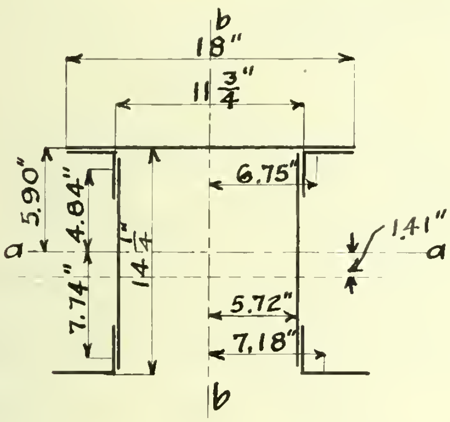
$$f_w = \frac{\frac{1}{8} \times 1,374 \times 234 \times 7.39}{457.9 - \frac{315,300 \times 234^2}{10 \times 28,000,000}} = -\ 320 \text{ pounds (compression).}$$

The unit stress due to the weight of the member and eccentric loading is, therefore, $\frac{1,530 + 320}{13,040} \times 100 = 14.2$ percent and must be considered.

Hence the actual unit stress due to dead, live, and wind loads, eccentric loading, and weight is

$P = 13{,}040 + 6{,}070 + 1{,}530 + 320 = 20{,}960$ pounds per square inch, while the average allowable unit stress due to the same loads is $13{,}040 + 0.25 \times 13{,}040 + 0.10 \times 13{,}040 = 17{,}600$ pounds per square inch, which gives an efficiency of $\frac{17{,}600}{20{,}960} = 0.877$.

5(e). Section $U_1'U_3'$. This section of the short truss of the 270 foot span is the same as the corresponding section of of the long truss, section U_1 to U_3, except that $\frac{5}{16}$-inch web plates are used in place of the $\frac{3}{8}$-inch plates of that section. The section is shown by Fig. 20. It is composed of two angles $3" \times 3" \times \frac{5"}{16}$, two angles $3" \times 4" \times \frac{7"}{16}$, two web plates $14" \times \frac{5"}{16}$, and one cover plate $18" \times \frac{3"}{8}$. The greatest stresses carried by the member are 162,100 pounds due to dead load, 169,000 pounds due to live load, and 91,500 pounds due to wind load.



The area of the section is as follows,

Fig. 20.

2- Ls-	$3" \times 3" \times \frac{5"}{16}$	= 3.56	square inches
2- "	$3" \times 4" \times \frac{7"}{16}$	= 5.74	" "
2 Web Pl.	$14" \times \frac{5"}{16}$	= 8.75	" "
1 Cov. "	$18" \times \frac{3"}{8}$	= 6.75	" "
Total		= 24.80	square inches.

The static moment of the section about the top line of the cover plate is,

Moment of cover plate = 6.75 x 0.19 = 1.3 inches^3

" " 2 web " = 8.75 x 7.50 = 65.7 "

" " 2- ^{S}-3"x 3" = 3.56 x 1.25 = 4.5 "

" " 2- " 3"x 4" = 5.74 x 13.8 = 79.4 "

Total = 150.9 inches^3.

The neutral axis is $\frac{150.9}{24.80}$ = 6.09 inches below the top of the section giving an eccentricity of 7.50 - 6.09 = 1.41 inches.

The moment of inertia about the axis a-a is,

Moment of angles = $2(1.51 + 1.78 \times 4.84^2)$ = 86.8 inches^4

" " " = $2(2.18 + 2.87 \times 7.74^2)$ = 348.8 "

" " web pl.= $2(71.46 + 4.38 \times 1.4^2)$ = 160.3 "

" " Cov. " = $0.08 + 6.75 \times 5.90^2$ = 235.1 "

I_{a-a} = 831.0 inches^4,

and the radius of gyration, $r_{a-a} = \sqrt{\frac{831.0}{24.80}}$ = 5.78 inches.

The unsupported length of the member being 234 inches the allowable dead load stress is $22,000 - 80 \times \frac{234}{5.78}$ = 18,760 pounds per square inch and for live load the allowable stress is $\frac{18,760}{2}$ = 9,380 pounds per square inch.

The required areas are $\frac{162,100}{18,760}$ = 8.62 square inches and $\frac{169,000}{9,380}$ = 18.00 square inches for dead and live load respectively.

The average allowable unit stress for dead and live loads is $\frac{162,100 + 169,000}{8.62 + 18.00}$ = 12,440 pounds per square inch, while the actual unit stress produced by these loads is $\frac{162,100 + 168,000}{24.80}$ = 13,340 pounds per square inch. The unit stress from the wind load is $\frac{91,500}{24.8}$ = 3,690 pounds per square inch and this is $\frac{3,690}{13,340} \times 100$ = 27.7 percent of the average stress due to dead and live loads and must be considered.

The stress in the lower fibre due to eccentricity of longitud-

dinal loading is

$$f_e = \frac{422,600 \times 1.41 \times 8.52}{831.0 - \frac{422,600 \times 234^2}{10 \times 28,000,000}} = -6,780 \text{ pounds (compression)}$$

The weight of the unsupported length of the member is,

2- 3"x 3"x $\frac{5"}{16}$ angles	=	238 pounds
2- 3"x 4"x $\frac{7"}{16}$ "	=	382 "
2 Web Pl. 14"x $\frac{5"}{16}$	=	580 "
1 Cov. " 18"x $\frac{3"}{8}$	=	448 "
Details and Lacing	=	402 "
Total Weight	=	2,050 pounds.

The stress in the lower fibre due to the weight of the membeb is

$$f_w = \frac{\frac{1}{8} \times 2,050 \times 234 \times 8.52}{831.0 - \frac{422,600 \times 234^2}{10 \times 28,000,000}} = +680 \text{ pounds (tension).}$$

The unit stress due to weight and eccentric loading of the member is, therefore, $\frac{6,780 - 680}{13,340} = 45.7$ percent and must be considered.

Hence the actual unit stress in the member due to dead, live, and wind loads, eccentric loading, and weight, is

$P = 13,340 + 3,690 + 6,780 - 680 = 23,130$ pounds per square inch, while the average allowable unit stress due to the same loads is $12,440 + 0.25 \times 12,440 + 0.10 \times 12,440 = 16,820$ pounds per square inch, giving an efficiency of $\frac{16,820}{23,130} = 0.728$.

5(f). Section $U_3'U_5'$. This section of the upper chord of the short truss of the 270 foot span is composed of two angles 3"x 3"x$\frac{3"}{8}$, two angles 3"x 4"x $\frac{3"}{8}$, two web plates 14"x $\frac{7"}{16}$, and one cover plate 18"x $\frac{3"}{8}$. The section is shown by Fig. 21. The greatest stresses carried by the member are 187,100 pounds, 195,000 pounds, and 103,000

pounds due to dead, live, and wind loads respectively.

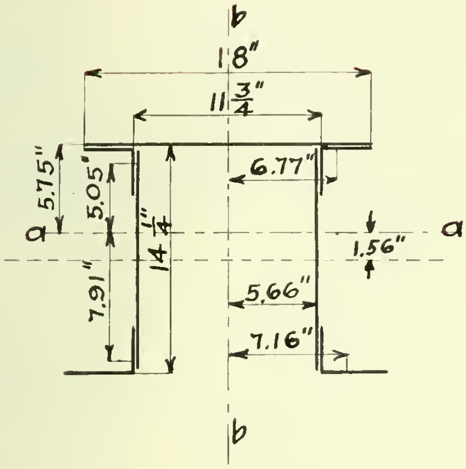


Fig. 21.

The area of the section is as follows,

2-$\angle$s- 3"x 3"x $\frac{3"}{8}$	= 4.22	square inches	
2- " 3"x 4"x $\frac{3"}{8}$	= 4.96	"	"
2 Web PL.14"x $\frac{7"}{16}$	= 12.25	"	"
1 Cov. " 18"x $\frac{3"}{8}$	= 6.75	"	"
Total	= 28.18	square inches.	

The static moment of the section about the top line of the cover plate is,

Moment of cover plate	= 6.75 x 0.19	=	1.3 inches^3
" " 2 web "	= 12.25 x 7.50	=	91.4 "
" " 2- $\angle$s- 3"x 3"x $\frac{3"}{8}$	= 4.22 x 1.27	=	5.4 "
" " 2 " 3"x 4"x $\frac{3"}{8}$	= 4.96 x 13.85	=	68.7 "
	Total	=	167.3 inches^3

The neutral axis is therefore $\frac{167.3}{28.18}$ = 5.94 inches below the top of the section and this gives an eccentricity of 7.50 - 5.94 = 1.56 inches.

The moment of inertia about the axis a-a is,

Moment of angles	= $2(1.76 + 2.11 \times 5.05^2)$	= 111.2 inches^4
" " "	= $2(1.92 + 2.48 \times 7.91^2)$	= 315.0 "
" " web pl.	= $2(100.04 + 6.13 \times 1.56^2)$	= 229.9 "
" " cov. "	= $0.08 + 6.75 \times 5.75^2$	= 224.3 "
	I_{a-a}	= 880.4 inches^4,

and the radius of gyration, $r_{a-a} = \sqrt{\frac{880.4}{28.18}}$ = 5.59 inches.

As the unsupported length of the member is 234 inches the allowable dead load stress is 22,000 - 80 x $\frac{234}{5.59}$ = 18,650 pounds per

square inch and for live load $\frac{18,650}{2}$ = 9,330 pounds per square inch.

The required areas are $\frac{187,100}{18,650}$ = 10.02 square inches and $\frac{195,000}{9,330}$ = 20.90 square inches for dead and live load respectively. The average allowable stress for dead and live loads is $\frac{187,100 + 195,000}{10.02 + 20.90}$ = 12,350 pounds per square inch. The actual stress in the member due to these loads is $\frac{187,100 + 195,000}{28.18}$ = 13,540 pounds per square inch. The stress due to the wind is $\frac{103,000}{28.18}$ = 3,650 pounds per square inch and as this is $\frac{3,650}{13,540}$ x 100 = 26.9 percent of the average stress due to dead and live loads it must be considered.

The stress in the lower fibre due to eccentricity is

$$f_e = \frac{485,100 \times 1.56 \times 8.69}{880.4 - \frac{485,100 \times 234^2}{10 \times 28,000,000}} = -\ 8,380 \text{ pounds.}$$

The weight of the unsupported length of the member is,

2- 3"x 3"x $\frac{3}{8}$" angles	=	282	pounds
2- 3"x 4"x $\frac{3}{8}$" "	=	332	"
2- Web Pl. 14"x $\frac{7}{16}$"	=	813	"
1- Cov. " 18"x $\frac{3}{8}$"	=	446	"
Details and Lacing	=	468	"
Total Weight	=	2,343	pounds.

The stress in the lower fibre due to the weight of the member is

$$f_w = \frac{\frac{1}{8} \times 2,343 \times 234 \times 8.69}{880.4 - \frac{485,100 \times 234^2}{10 \times 28,000,000}} = +\ 760 \text{ pounds.}$$

The unit stress due to eccentric loading and the weight of the member is, therefore, $\frac{8,380 - 760}{13,540}$ x 100 = 56.2 percent of the average allowable stress due to dead and live loads and must be considered.

Hence the actual unit stress in the member due to dead, live, and wind loads, eccentric loading, and weight, is

$P = 13{,}540 + 3{,}650 + 8{,}380 - 760 = 24{,}800$ pounds per square inch, while the allowable unit stress is

$P' = 13{,}540 + 0.25 \times 13{,}540 + 0.10 \times 13{,}540 = 18{,}300$ pounds per square inch giving an efficiency of $\frac{18{,}300}{24{,}800} = 0.738$.

Table X is a summary of the investigation of the compression members. The stresses in column 4 include the wind load stresses, in those cases where they exceed 25 percent of the sum of the dead and live load stresses, as required by the Specifications.

In reference number 4 the maximum compressive stress occurs in member L_2U_3 but there is no reversal of stress in that member. The reversal of stress occurs in member U_3L_4, and the tension stress as given in column 4 has been increased by eight-tenths of the smaller stress as required.

An examination of Table X shows that all upper chord sections are inefficient, while the connections of all memoers except the end posts, U_0L_0 and U_7L_7, and the sub-verticals, Mn, are inefficient. The inefficiency of member 4 for tension is due to the method of makeing the end connections, the angles composing the members being connected by the smaller leg only, thereby giving a very small efficient net area.

The last three members given in the table, 12, 13 and 14, are members of the short truss, they being of a little different design than the corresponding members of the long truss. The remainder of the members of the short truss are the same as the corresponding members of the long truss, and since the stresses were lighter than those of the long truss they were not investigated.

Table X.

Compression Members.

Ref. No.	Member	Section Area Sq. In.	Total Stress* Lbs.	Unit Stresses Lbs. per Sq. In.		Efficiencies of	
				Actual	Average Allowable	Mem-bers.	Connect-ions
1	U_0L_0	16.76	154,400	9,200	13,900	1.51	0.67
2	U_7L_7	25.72	301,300	11,700	15,600	1.33	1.17
3	L_0U_1	34.21	398,500	13,676	15,800	1.15	1.08
4	L_2U_3, U_3L_4	13.24	111,600 †88,200	8,400 23,700	11,600 13,400	1.38 0.56	0.74
5	U_5L_6	20.92	194,500	9,300	12,100	1.19	0.70
6	U_0U_1	4.76	16,600	3,500	12,600	3.61	
7	U_1U_3	26.55	439,700	22,200	16,700	0.75	
8	U_3U_5	32.47	414,200	15,100	12,400	0.90	
9	U_5U_7	16.49	315,300	21,000	17,600	0.88	
10	Intermediate Posts L U	8.36	62,300	7,500	11,800	1.45	0.87
11	Sub-Verticals M n	5.70	28,300	5,000	9,400	1.90	1.00
12	$L_0'U_1'$	32.21	386,700	15,660	15,800	1.01	0.91
13	$U_1'U_3'$	24.80	422,600	23,100	16,800	0.73	
14	$U_3'U_5'$	28.18	485,100	24,800	18,300	0.74	

*Where the wind load stress in a member exceeds 25 percent of the combined dead and live load stresses in the same member it is included in the total stress, otherwise not.

†Tension.

Tension Members.

In investigating the tension members the stress in the bars due to their own weight will be considered. This stress is computed from the formula

$$f_w = \frac{My}{I+\frac{Pl^2}{10E}}$$

the nomenclature being as follows:

f_w = fibre stress due to cross bending,

M = bending moment = $\frac{1}{8}wl^2$,

I = moment of inertia of the member,

P = total direct stress in the member,

Y = distance from neutral axis to remote fibre,

l = length of member in inches, and

E = modulus of elasticity of the material.

Also let b = breadth of member in inches,

d = depth of member in inches = $2Y$,

f_p = direct fibre stress = $\frac{P}{bd}$, and

w = weight of bar per lineal inch = $0.278\ bd$.

Now $P = f_p bd$, $I = \frac{bd^2}{12}$, and $E = 28{,}000{,}000$.

Substituting these in the formula we have

$$f_w = \frac{\frac{1}{8}wl^2\ \frac{d}{2}}{\frac{bd^3}{12}+\frac{f_p bdl^2}{10\text{x}28{,}000{,}000}}$$, which reduces to

$$f_w = \frac{4{,}900{,}000\ d}{f_p+23{,}000{,}000\left(\frac{d}{e}\right)^2}$$

An examination of Plates II and III shows that all bars except the sub-ties are 5 inches deep. For lower chord members the length, l, is 19 x 12 + 6 = 234 inches, and hence,

$$f'_w = \frac{4,900,000 \times 5}{f_p + 23,000,000\left(\frac{5}{234}\right)^2}$$

$$= \frac{24,500,000}{f_p + 100}$$

For wet members the length is

$l = \sqrt{19.5^2 + 20^2} \times 12 = 335$ inches. As these members are inclined, the stress obtained by the formula must be multiplied by the sine of the angle that the bar makes with a verticle line. The sine of this angle is $\frac{19.5 \times 12}{335} = 0.698$, hence

$$f''_w = \frac{4,900,000 \times 5 \times 0.698}{f_p + 23,000,000\left(\frac{5}{335}\right)^2}$$

$$= \frac{17,140,000}{f_p + 51}$$

These formulas will be used in obtaining the stresses in the tension members, but where the values of f_3 as given by them are less than ten percent of the allowed unit stress on the members they will not be considered.

The allowed unit strain for bottom chords, diagonals, and long verticles are 25,000 pounds per square inch for dead loads, and 12,500 pounds per square inch for live loads.

The method of procedure in the investigation of the tension members is analogous to that of the compression members. The

results are given in Tables XI and XII. Table XI containing the results for the long truss of the 270 foot span, and Table XII containing those for the short truss of the same span. Since the sub-ties, Un, and the suspenders, LU, are the same for both trusses they are omitted from Table XII.

Table XI.

Tension Members, Long Truss.

Ref. No.	Member	Section Area Sq. In.	Total Stress Lbs.	Unit Stresses Lbs. per Sq. In. Actual	Unit Stresses Lbs. per Sq. In. Average Allowable	Efficiency
1	Sub-Ties Un	1.76	24,200	13,800	16,500	1.20
2	U_1n_2	14.38	218,200	15,200	16,500	1.09
3	n_2L_2	12.50	195,000	15,600	16,400	1.05
4	L_4n_5	8.12	111,600	13,700	15,800	1.15
5	n_5U_5	9.38	133,700	14,300	15,900	1.11
6	L_6n_7	17.50	284,400	16,200	16,600	1.03
7	n_7U_7	19.38	307,800	15,900	16,600	1.04
8	L_oL_2	12.50	199,000	15,900	16,600	1.04
9	L_2L_4	23.00	396,300	15,900	16,600	1.04
10	L_4L_6	21.24	332,300	15,700	16,600	1.06
11	Suspenders LU	0.77	5,600	7,300	25,000	3.43

Table XII

Tension Members, Short Truss.

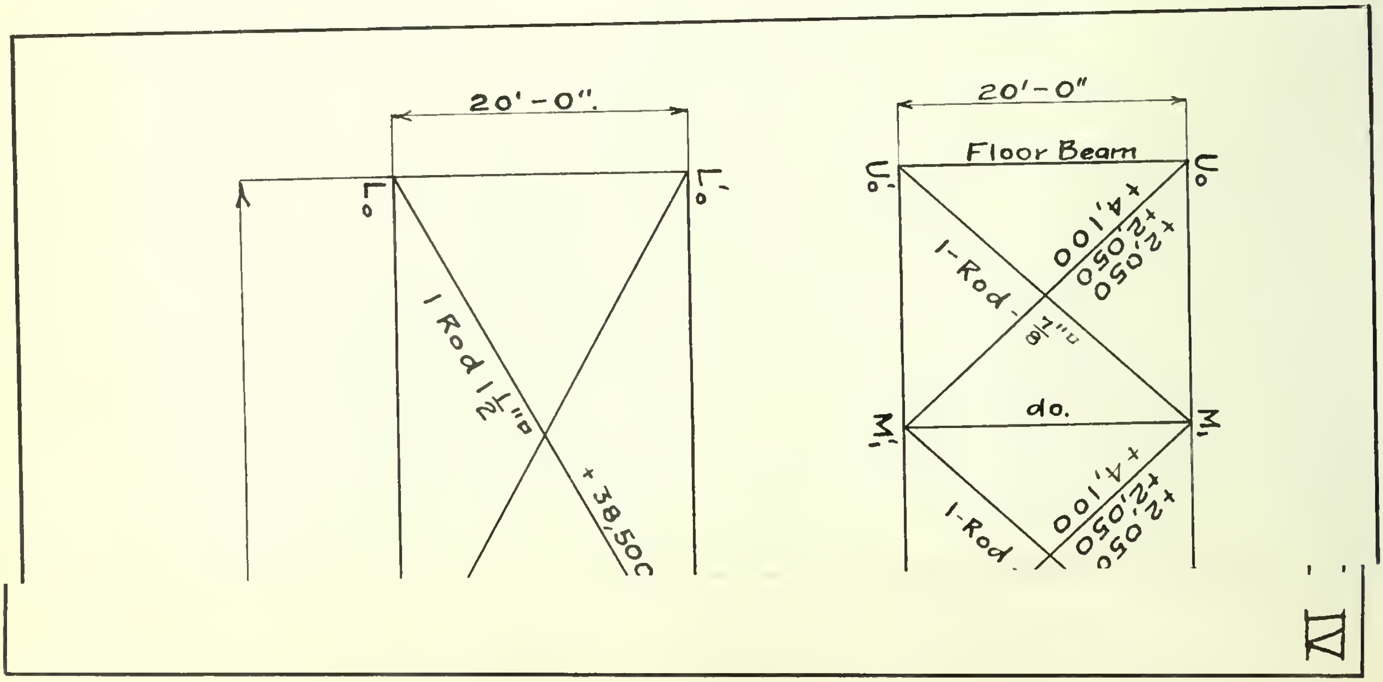
Ref. No.	Member	Section Area Sq. In.	Total Stress Lbs.	Unit Stresses Lbs. per Sq. In.		Efficiency
				Actual	Average Allowable	
1	$U_1'n_2'$	13.12	206,800	15,800	16,400	1.04
2	$n_2'L_2'$	11.88	183,100	15,400	16,40	1.06
3	$L_4'n_5'$	8.12	120,400	14,800	16,000	1.08
4	$n_5'U_5'$	9.38	144,100	15,400	16,000	1.04
5	$L_6'n_7'$	16.26	258,100	15,900	16,400	1.03
6	$n_7'U_7'$	17.50	279,300	16,000	16,400	1.02
7	$L_0'L_2'$	11.88	190,300	16,000	16,500	1.03
8	$L_2'L_4'$	23.14	372,300	16,100	16,500	1.02
9	$L_4'L_6'$	18.14	291,300	16,100	16,500	1.02

Upper and Lower Laterals.

Table XIII gives the results of the investigation of the lateral systems. The stresses and composition of the members of these lateral systems are shown in Plate IV.

The floor beams act as the upper lateral struts and their investigation is not necessary here. The lower lateral struts are all the same and, as Plate IV shows, the greatest stresses occur in L_1L_1' and L_6L_6'; as the stresses are the same in both only one need be investigated. This is the last member in Table XIII, number 17.

In Table XIV are given the results of the investigation of the sway bracing, the stresses and composition of the members being given on Plate V. Number 2 is the strut at the center of the inclined end posts L_0U_1 and $L_0'U_1'$, as shown on the plate. The stress in the member is compression while all the other members given in the table are in tension.


20'-0".
L₀
L'₀
1 Rod 1½"□
+38,500
20'-0"
Floor Beam
U'₀
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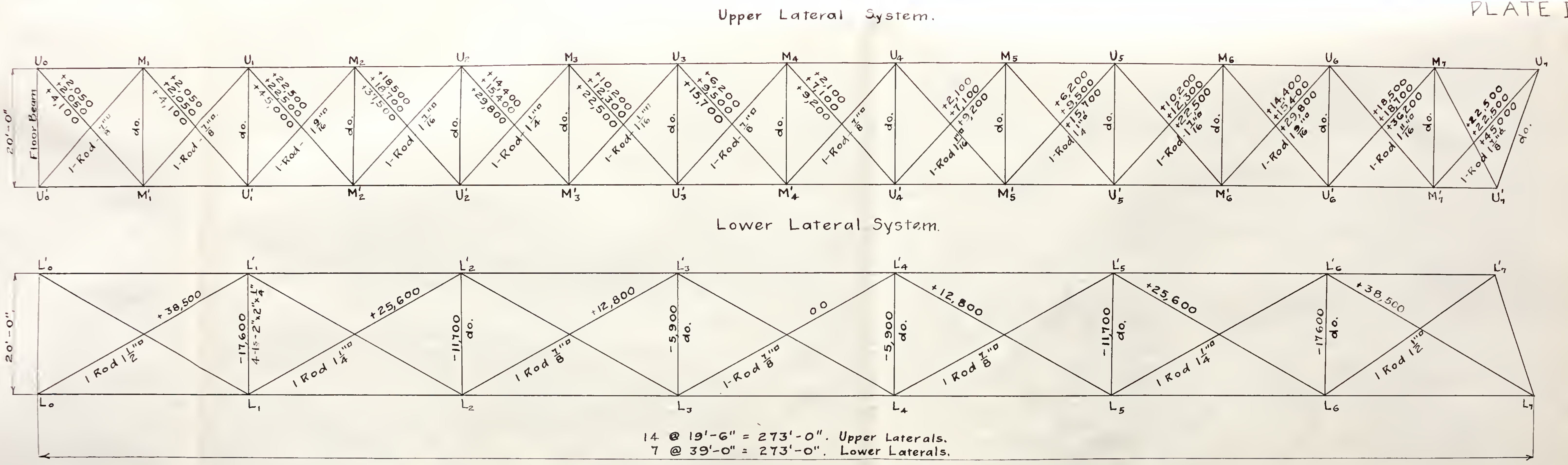
IV



UPPER AND LOWER LATERALS.

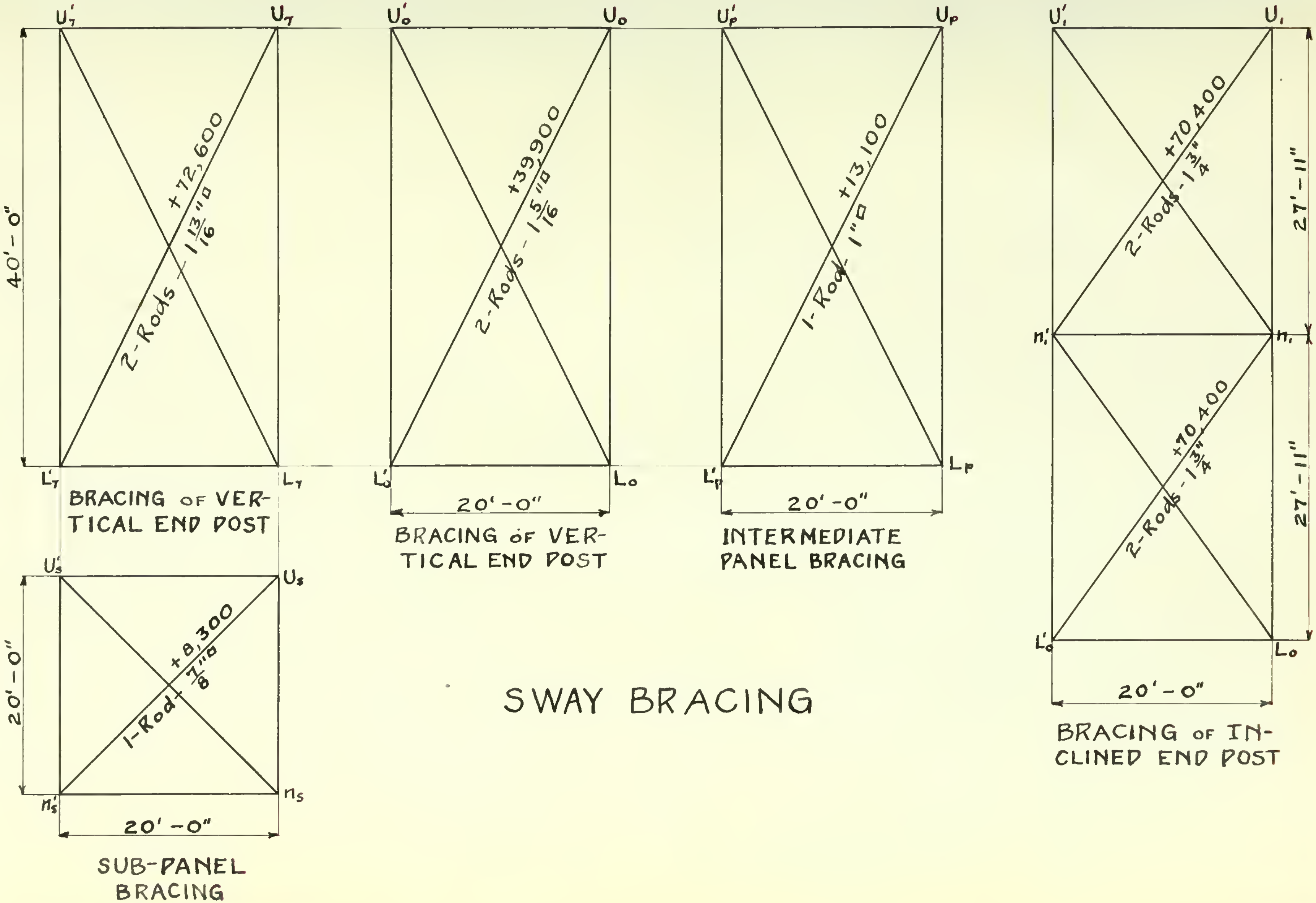
Table XIII.

Upper and Lower Laterals.

Ref. No.	Member	Section Area Sq. In.	Total Stress Lbs.	Unit Stresses Lbs. per Sq. In.		Efficiency
				Actual	Average Allowable	
1	U_oM_1'	0.77	4,100	5,300	18,000	3.40
2	U_1M_2'	2.44	45,000	18,400	"	0.98
3	M_2U_2'	2.07	37,500	18,100	"	0.99
4	U_2M_3'	1.56	29,800	19,100	"	0.94
5	M_3U_3'	1.13	22,500	19,900	"	0.91
6	U_3M_4'	0.77	15,700	20,400	"	0.88
7	M_4U_4'	0.77	9,200	11,900	"	1.51
8	U_4M_5'	1.13	9,200	8,100	"	2.22
9	M_5U_5'	1.56	15,700	10,100	"	1.78
10	U_5M_6'	2.07	22,500	10,900	"	1.65
11	M_6U_6'	2.44	29,800	12,200	"	1.47
12	U_6M_7'	2.85	36,200	12,700	"	1.42
13	M_7U_7'	3.51	45,000	12,800	"	1.40
14	L_oL_1'	2.25	38,500	17,100	"	1.05
15	L_1L_2'	1.56	25,600	17,000	"	1.06
16	L_2L_2'	0.77	12,800	16,600	"	1.08
17	L_1L_1'	3.76	17,600*	4,700	9,800	2.08

* Compression

PLATE V



SWAY BRACING

Table XIV.

Sway Bracing.

Ref. No.	Member	Section Area Sq. In.	Total Stress Lbs.	Unit Stresses Lbs. per Sq. In.		Efficiency
				Actual	Average Allowable	
1	$U'_k n'_k$	6.12	70,400	11,800	18,000	1.53
2	$n_1 n'_1$	4.76	40,350*	8,600	10,900	1.27
3	$L'_o U_o$	3.44	39,900	11,600	18,000	1.55
4	$L'_7 U_7$	6.56	72,600	11,100	"	1.62
5	$L_p U'_p$	1.00	13,100	13,100	"	1.37
6	$n_s U'_s$	0.77	8,300	10,800	"	1.67

* Compression.

Joists.

In the determination of the live load for which the floor was designed on page 15, it was found that the maximum dead load moment in the beam is 27,600 pound inches and that the section modulus of the beam is 130.

The joists are spaced about 2 feet 6 inches center to center, and the panel length is 19 feet 6 inches center to center of floor beams, hence with a live load of 80 pounds per square foot, as specified for a class D bridge, the total live load supported by one joist is 2.5 x 19.5 x 80 = 3,900 pounds. The maximum moment due to the live load is $\frac{1}{8}$ x 3,900 x 19.5 x 12 = 114,400 pound inches. The total moment in the beam is, therefore, 114,400 + 27,600 = 142,000 pound inches, which gives a stress of $\frac{142,000}{130}$ = 10,900 pounds per square inch and as the allowable stress is 12,000 pounds per square inch the efficiency is $\frac{12,000}{10,900}$ = 1.10.

The alternate load for a class D bridge is 6 tons or 12,000 pounds in two axles 10 feet centers. This gives a concentrated load of $\frac{12,000}{4}$ = 3,000 pounds on each wheel. The load will be considered to be divided equally among four joists so that the load on each joist is 3,000 pounds applied at two points, 10 feet apart. The maximum moment in the beam will occur under one wheel when they are placed so that the center of gravity of the two loads is as far to one side of the middle as the point of maximum moment is to the other side of the middle. Hence the wheel under which the maximum moment will occur should be $\frac{19.5}{2} - \frac{10}{4}$ = 7.25 feet from one end of the beam. The position of the wheels is illustrat-

ed in Fig. 23.

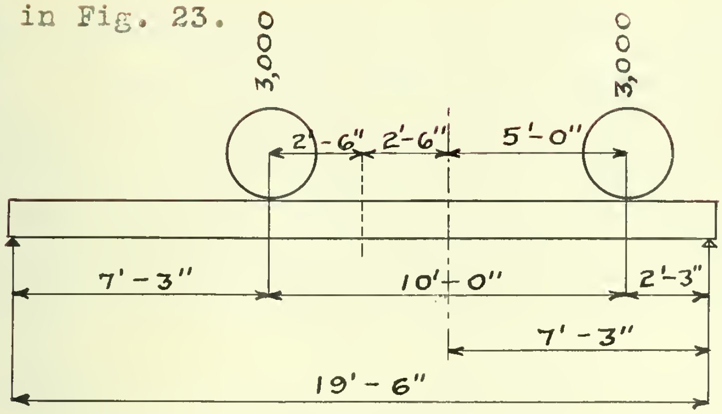


Fig. 23.

With the loads and positions shown in the figure the left reaction is $\frac{3,000 \times 7.25}{19.5} = 1,120$ pounds and the maximum moment is

$1,120 \times 7.25 \times 12 = 97,100$ pound inches.

The weight of floor and joist per linear foot is $(2.5 \times 2.5 + 1.17 \times 4)4.5 = 49.2$ pounds. The left reaction is $49.2 \times \frac{19.5}{2} = 480$ pounds, and the moment at the dangerous section due to deal load is $(480 \times 7.25 - 49.2 \times \frac{7.25^2}{2})\ 12 = 76,200$ inch pounds. The total moment at the section is, therefore, $26,200 + 97,100 + 123,300$ pound inches and the stress in the beam is the $\frac{123,300}{130} = 9,500$ pounds per square inch, giving an efficiency of $\frac{12,000}{9,500} = 1,264$.

Floor Beams.

The floor beams are 15 inch 50 pound steel I beams. The beams are contunuous over the trusses so that there is a 6 foot cantilever arm at each end as shown in Fig. 24. From Table I, the total weight of lumber on the span is 197,350 pounds, and the

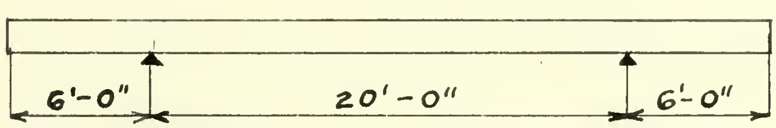


Fig. 24.

total supported by one floor beam is $\frac{197,350}{14}$ = 14,100 pounds. It will be sufficiently exact to consider this as distributed uniformly in the beams and as the weight of the beam is 50 pounds per lineal foot the total dead load on the beam is 14,100 + 50 x 32 = 15,700 pounds, hence the reaction at each end is $\frac{15,700}{2}$ = 7,850 pounds.

The dead load per lineal foot of beam is $\frac{15,700}{32}$ = 491 pounds. The greatest negative moment, due to deal load, is over the support and is $-491 \times \frac{6^2}{2} \times 12 = -106,000$ pound inches. The greatest dead load position moment is at the center of the beam and is $+ 7,850 \times 10 \times 12 - 491 \times \frac{16^2}{2} \times 12 = + 188,500$ pound inches.

At 80 pounds per square foot the live load on the beam is 80 x 19.5 = 1,760 pounds per lineal foot. The greatest live load negative moment occurs over the support when the cantilever arm is loaded and is $- 1,760 \times \frac{6^2}{2} \times 12 = - 380,000$ pound inches. The greatest live load positive moment occurs at the center of the beam when only that portion of the beam between the supports is loaded and is $+ \frac{1}{8} \times 1,760 \times 20^2 \times 12 = + 1,055,000$ pound inches.

The maximum moment occurs at the center of the beam, therefore, and is 1,055,000 + 188,500 = 1,243,500 pound inches. From Carnegie's hand book we get the section modulus of a 15-inch 50 pound I beam to be 64.5, therefore the stress in the beam is $\frac{1,243,500}{64.5}$ = 19,300 pounds per square inch. As the allowable stress is 13,000 pounds per square inch the efficiency is $\frac{13,000}{19,300}$ = 0.673.

As it is evident that the alternate load of 12,000 pounds will not give as great a moment as the uniform live load it will not be necessary to investigate the beam for it.

Art. 7 - 80-Foot Span.

The stresses and the composition of the members of the 80-foot span are given on Plate VI. The results of the investigation for different members are given in Table XV. The method of investigation was the same as that for the 270-foot span of Article 6.

Art. 8 - 60-Foot Span.

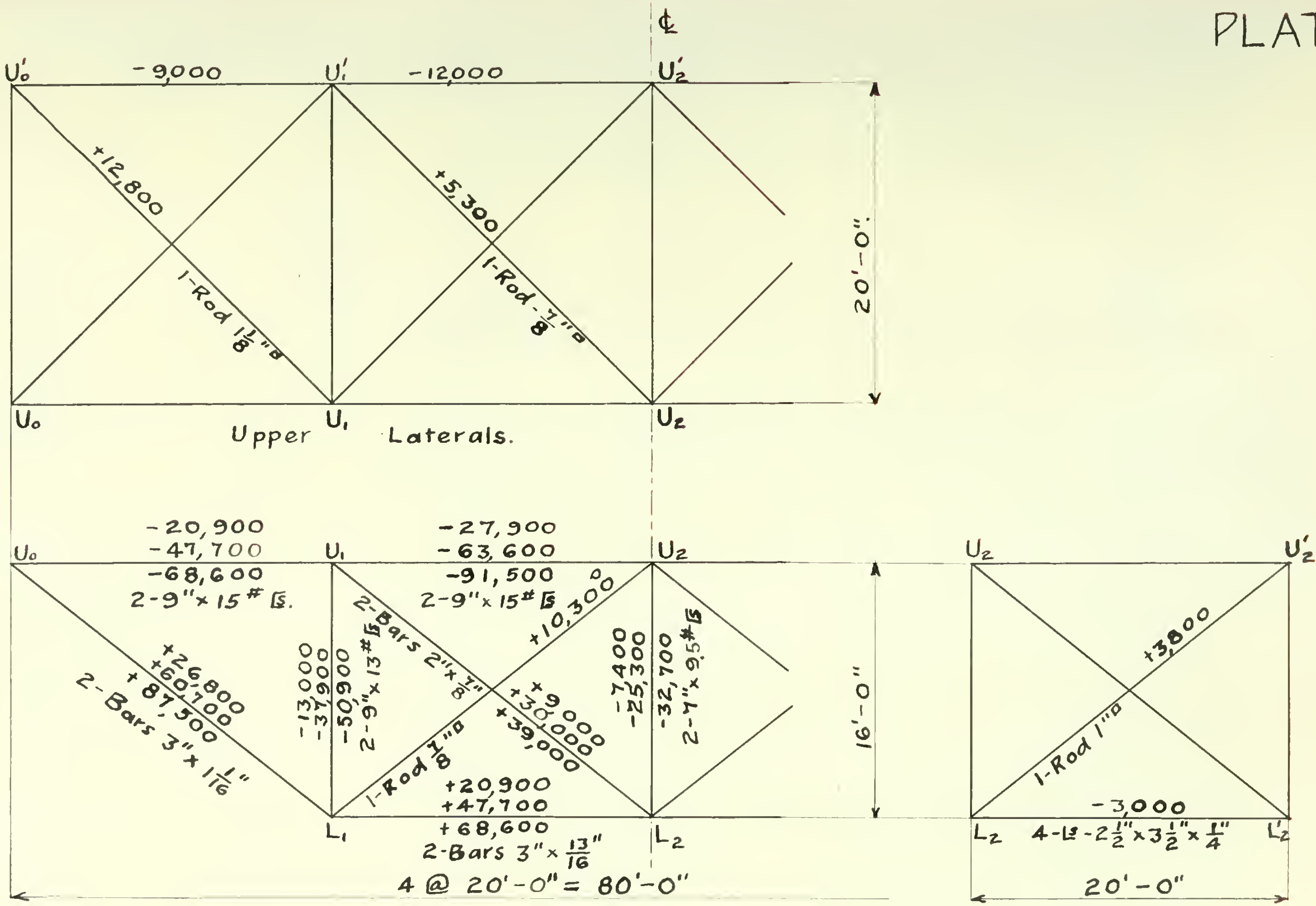
The stresses and the composition of the members of the 60-foot span are given on Plate VII, and the results of the investigation are given in Table XVI. The method of procedure was the same as for the former spans.

Art. 9 - Towers and Bents.

The stresses and the composition of the members of the towers and bents are given on Plates VIII to XII inclusive. The results of the investigations are given in Table XVII.

The longitudinal struts of all the towers are the same as the upper transverse struts of towers 1 and 4, and since the stresses in them are small they were not investigated.

PLATE VI



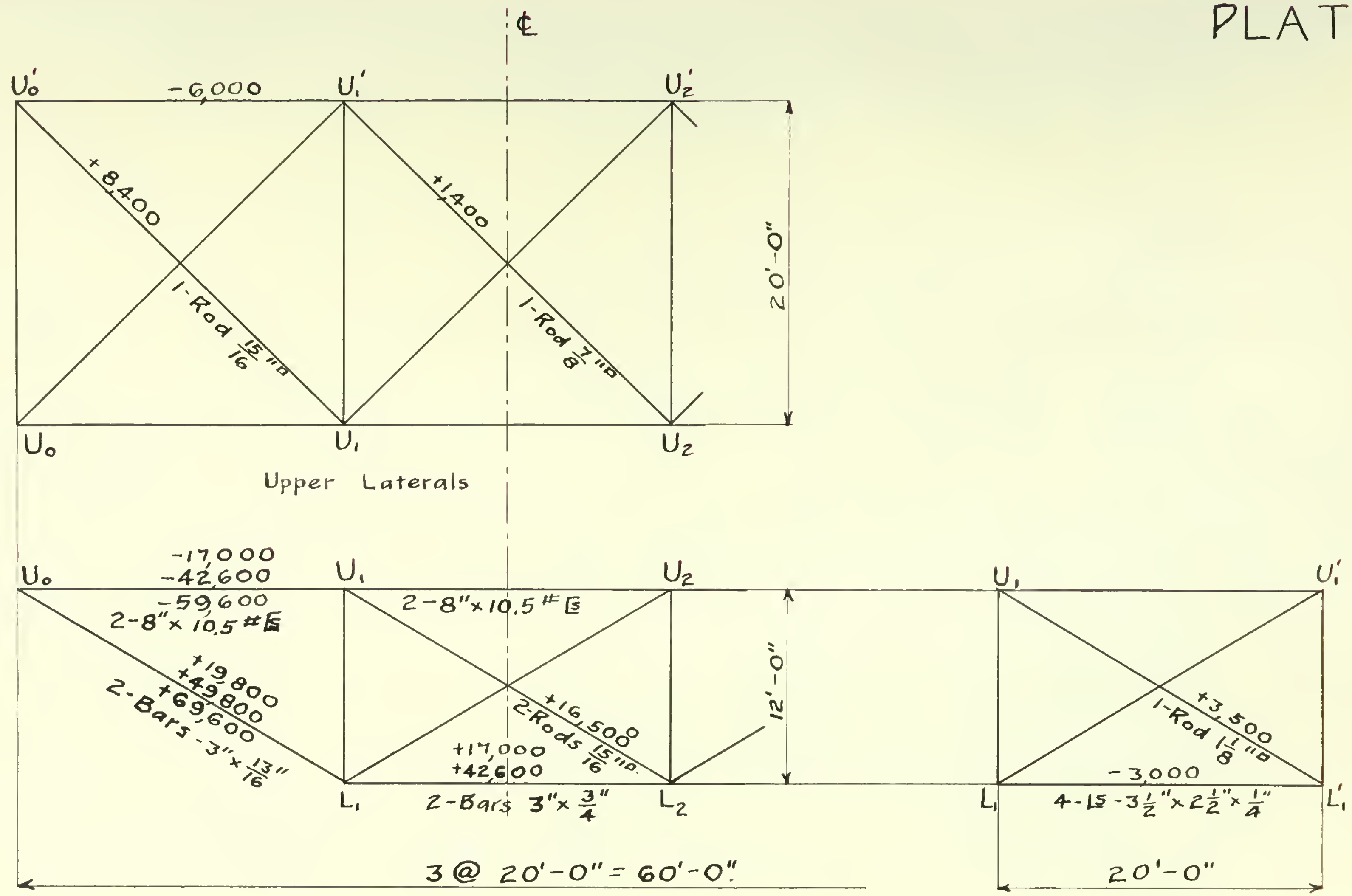
80-FOOT TRUSS

Table XV.

80-Foot Truss

Ref. No.	Member	Section Area Sq. In.	Total Stress Lbs.	Unit Stresses Lbs. per Sq. In.		Efficiencies of	
				Actual	Average Allowable	Members.	Connections
1	L_1U_1	7.78	50,900	6,500	10,100	1.55	1.29
2	L_2U_2	5.70	32,700	5,700	9,200	1.61	1.45
3	U_1U_2	8.82	91,500	11,600	11,400	0.98	1.39
4	U_oL_1	6.38	87,500	13,700	14,700	1.074	
5	U_1L_2	3.50	39,400	11,300	14,100	1.25	
6	L_1U_2	0.77	10,300	13,400	12,500	0.93	
7	L_1L_2	4.88	68,600	14,100	14,700	1.04	
8	$U_oU'_1$	1.27	12,800	10,100	18,000	1.78	
9	$U_1U'_2$	0.77	5,300	6,900	18,000	2.61	
10	$L_2U'_2$	1.00	3,800	3,800	18,000	4.74	
11	$L_2L'_2$	5.76	3,000	520	4,800	9.24	10.02

PLATE VII



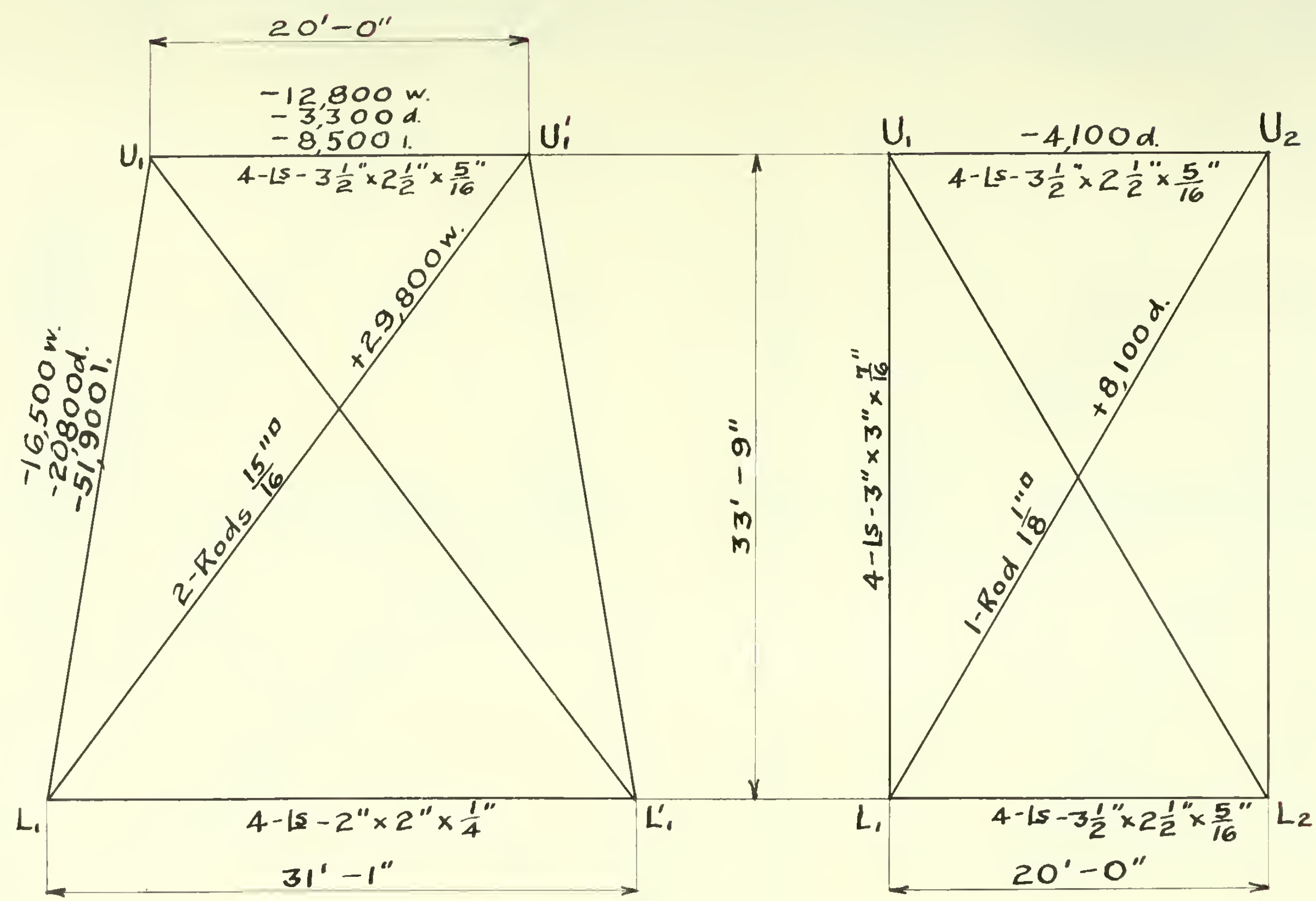
60-FOOT TRUSS

Table XVI

60-Foot Truss

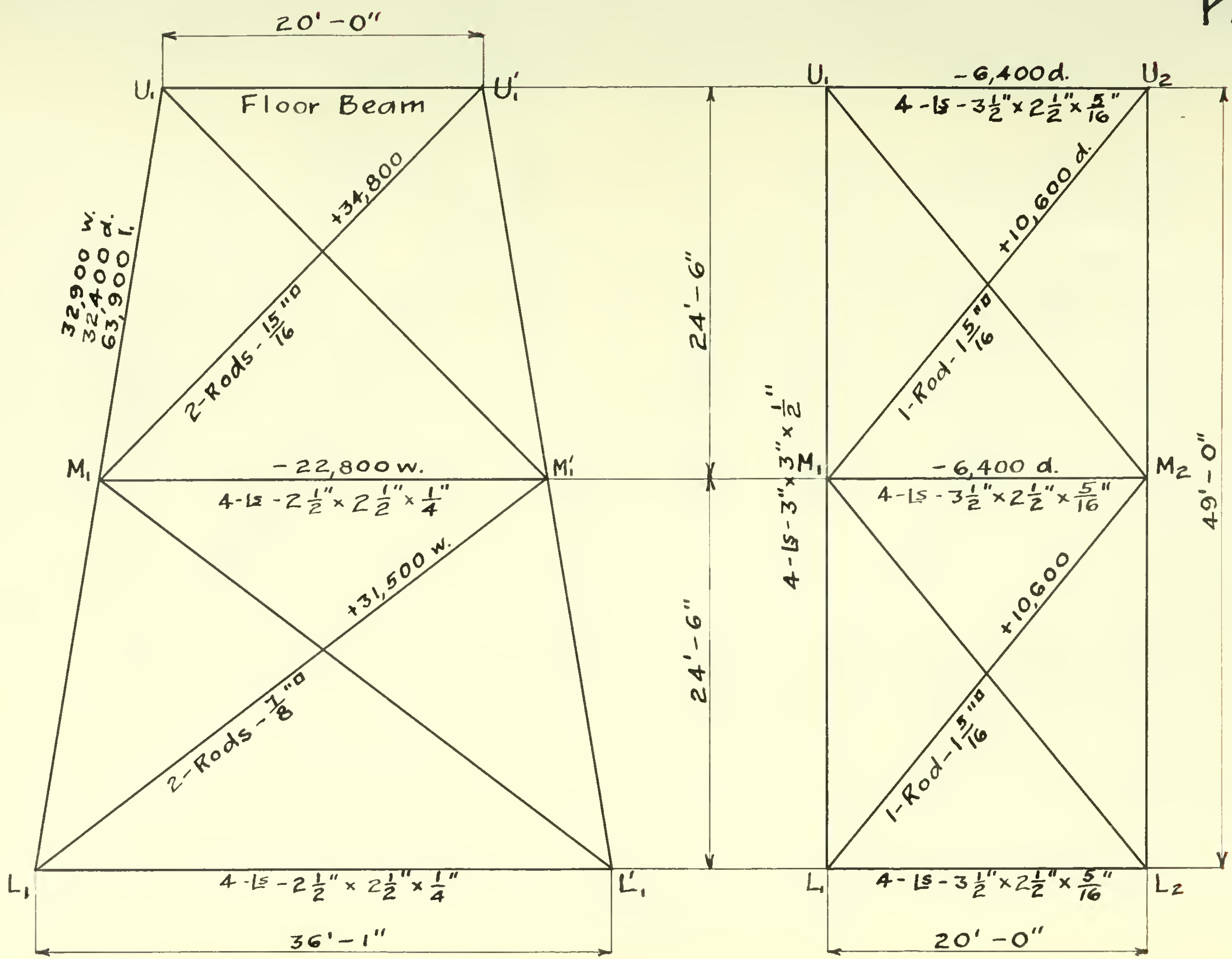
Ref. No.	Member	Section Area Sq. In.	Total Stress Lbs.	Unit Stresses Lbs. per Sq. In.		Efficiencies of	
				Actual	Average Allowable	Member	Connection
1	L_1U_1	5.36	35,800	6,700	9,900	1.47	1.03
2	U_1U_2	6.20	59,600	10,900	10,000	0.92	1.27
3	U_0L_1	4.88	69,600	14,200	14,600	1.03	
4	U_1L_2	1.76	16,500	9,400	12,600	1.33	
5	L_1L_2	4.50	59,600	13,200	14,600	1.10	
6	U_0U_1'	0.88	8,400	9,500	18,000	1.90	
7	U_1U_2'	0.77	1,400	1,800	18,000	10.00	
8	L_1U_1'	1.26	3,500	2,800	18,000	6.43	
9	L_1L_1'	5.76	3,000	520	4,800	9.24	10.02

PLATE VIII



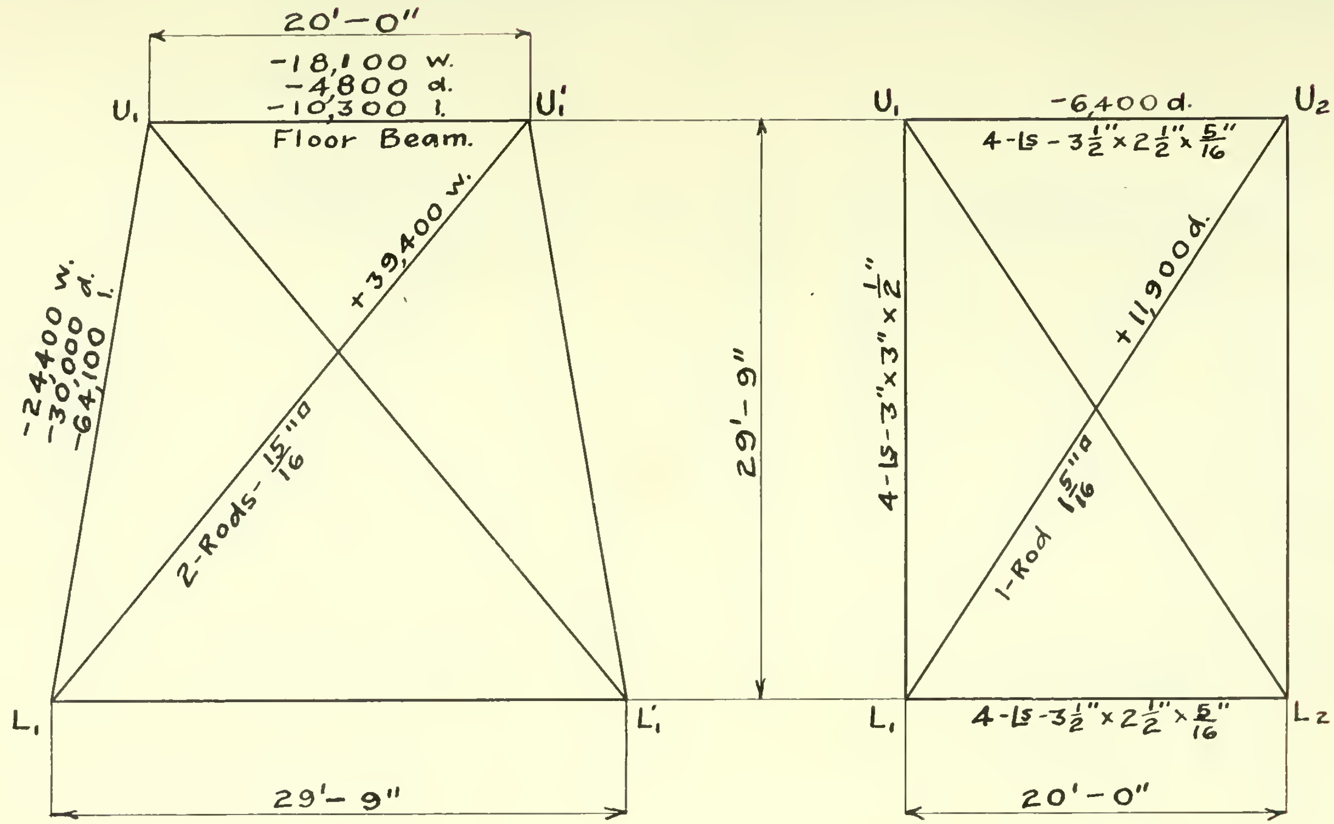
TOWER NO. 1.

PLATE IX



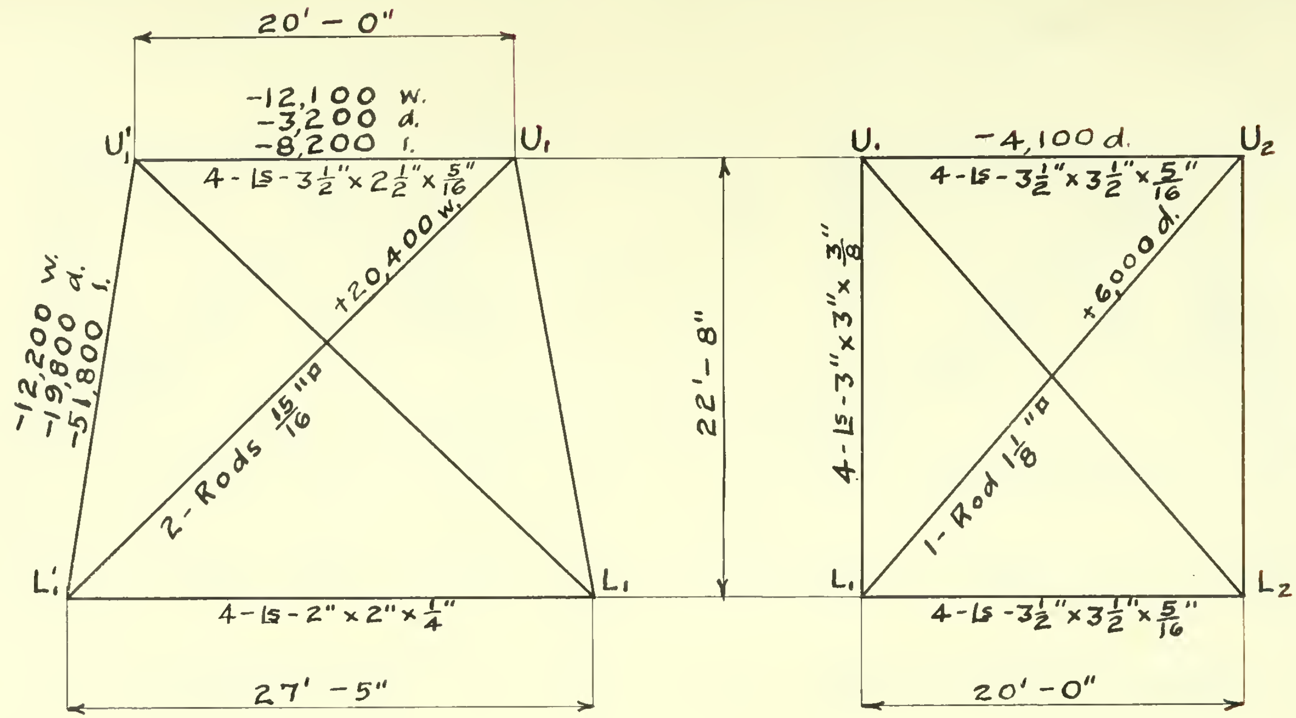
TOWER NO. 2.

PLATE X



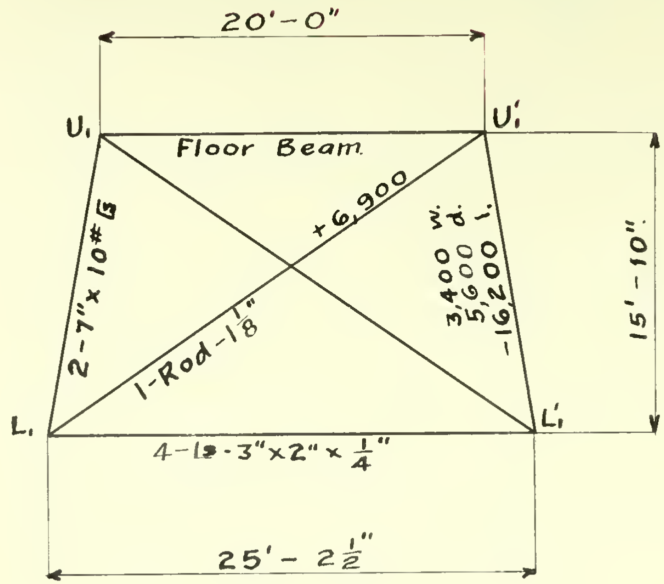
TOWER NO. 3.

PLATE XI

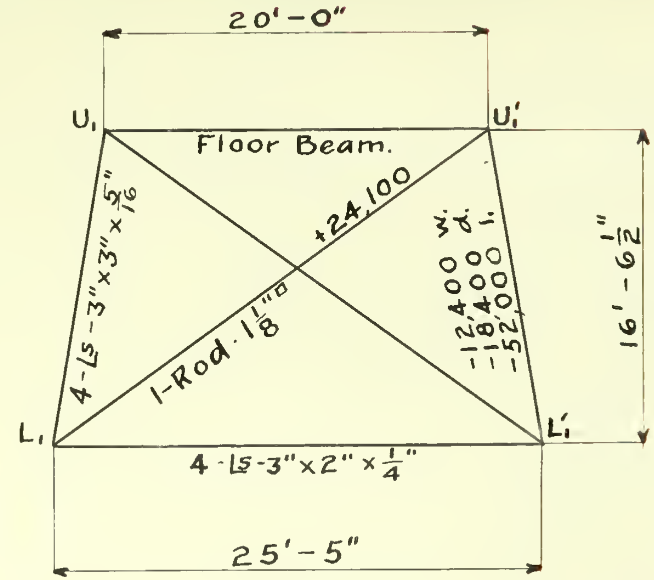


TOWER NO. 4.

PLATE XII



Bent No.1.



Bents No. 2 and 3.

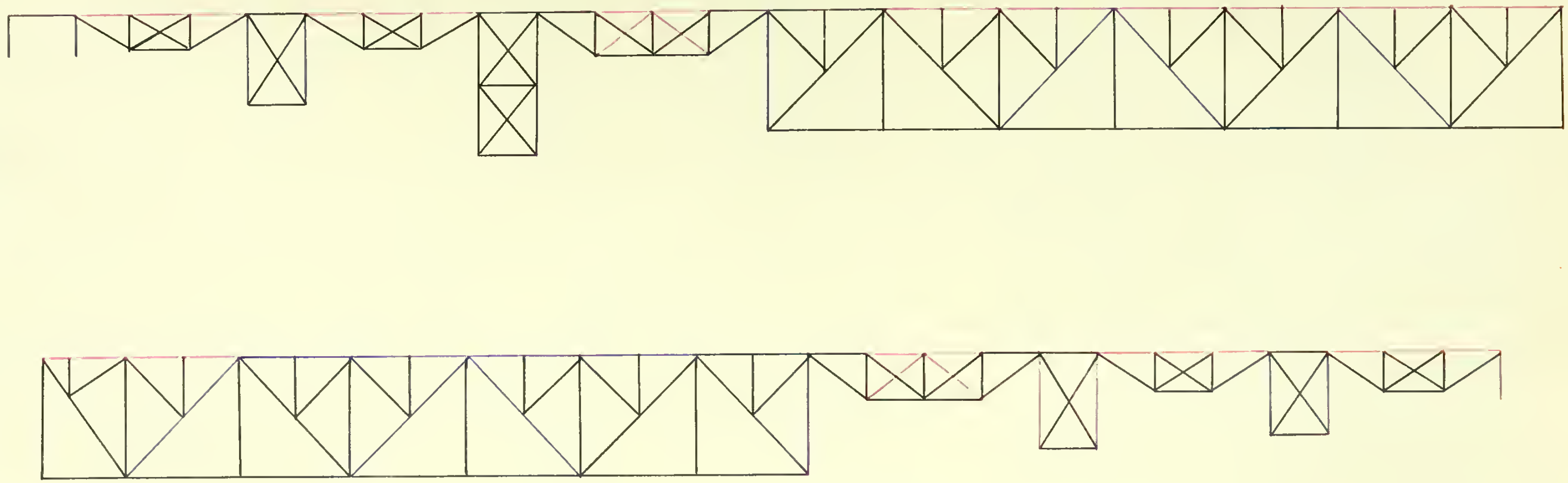
BENTS

Table XVII

Towers and Bents

Ref. No.	Member	Section Area Sq. In.	Total Stress Lbs.	Unit Stresses Lbs. per Sq. In.		Efficiencies of	
				Actual	Average Allowable	Member	Connection
	Tower 1						
1	L_1U_1	9.72	72,700	7,500	10,200	1.36	0.64
2	U_1U_1'	7.12	24,600	3,500	4,900	1.40	0.69
3	L_1U_1'	1.76	29,800	16,900	18,000	1.06	
4	L_1U_2	1.26	8,100	6,400	18,600	2.81	
	Tower 2						
5	L_1U_1	11.00	129,200	11,800	13,100	1.11	1.00
6	M_1M_1'	4.76	22,800	4,800	9,800	2.04	1.08
7	M_1U_1'	1.76	34,000	19,300	18,000	0.93	
8	L_1M_1'	1.53	31,500	20,500	18,000	0.88	
9	M_1U_2	1.72	10,600	6,200	18,000	2.91	
	Tower 3						
10	L_1U_1	11.00	118,500	10,800	13,000	1.20	0.94
11	L_1U_1'	4.76	39,400	8,300	18,000	2.17	
12	L_2U_2	1.72	11,900	6,900	18,000	2.61	
	Tower 4						
13	L_1U_1	8.44	71,600	8,500	10,600	1.25	0.65
14	U_1U_1'	7.12	23,500	3,300	4,900	1.48	0.72
15	L_1U_1'	1.76	20,400	11,600	18,000	1.55	
16	L_1U_2	1.26	6,000	4,800	18,000	3.75	
	Bent 1						
17	L_1U_1	5.70	21,800	3,800	9,400	2.47	1.87
18	L_1U_1'	1.27	6,900	5,400	18,000	3.33	
	Bent 2						
19	L_1U_1	7.12	70,400	9,900	11,400	1.15	0.65
20	L_1U_1'	1.27	24,100	19,000	18,000	0.95	

PLATE XIII



NOTATION

Black: Efficiency of 1.0 or Greater.
Red : " between 1.0 and 75%.
Blue : " less than 75%.

GRAPHIC REPRESENTATION
OF
EFFICIENCIES

Article 10-Conclusion.

A survey of the tables giving the results of the investigations of the preceeding pages shows that most of the main members of the structure are sufficient for the greatest stresses which they may have to carry. The most notable exceptions to the statement are the upper chords of the trusses of all the spans.

All segments of the upper chords are deficient in section area, the efficiencies running from 0.73 to 0.90 for the trusses of the long span, while for the 80-foot truss it is 0.98 and for the 60-foot truss it is 0.92.

In the case of the long trusses this inefficiency is mostly due to the eccentricity of the longitudinal loading because of the great flexural stress which it entails. Neglecting this flexural stress the efficiencies of the different segments of the chord would be much higher, averaging about 0.97.

Members U_1M_2', M_2U_2', U_2M_3', M_3U_3', and U_3M_4' of the upper lateral system of the long span are also somewhat deficient in area but this deficiency is not so serious as that of the upper chords.

The greatest deficiencies occur in the connections of the compression members. In several cases these connections are not nearly sufficient to develope the full strength of the member. This applies to both trusses of the 270-foot span and to the towers but not to either the 80- or the 60-foot trusses.

The efficiencies are represented graphically by the outline diagram of Plate XIII. Those members with an efficiency of unity or greater are shown in black, those with an efficiency between 75 percent and unity are shown in red, and those with an efficiency less than 75 percent are shown in blue.

As a whole the bridge will probably be sufficient for all the loads that will ever come upon it, since on account of its location it is not probable that it will ever receive the maximum loads that have beeb used in its investigation.